REPORT TO
INTERNATIONAL JOINT COMMISSION
ON
INVESTIGATION OF INTERNATIONAL
PASSAMAQUODDY TIDAL POWER PROJECT

APPENDIX 4

BASIC HYDROLOGIC DATA

BY

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INTERNATIONAL PASSAMAQUODDY
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INVESTIGATION OF INTERNATIONAL
PASSAMAQUODDY TIDAL POWER PROJECT

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4-01 PURPOSE

This appendix presents the hydrologic data essential to the studies of the proposed international Passamaquoddy tidal power project and the auxiliary river and pumped-storage power developments. The locations of the river and pumped-storage sites in relation to the location of the proposed tidal plant is shown on plate 4-1.

4-02 SCOPE

The basic hydrologic data reported on in this appendix include observed and computed tables of stages, stream flow, and water loss data; stage-discharge relationships; computed unit hydrographs; area-capacity curves; outline of drainage areas, ponding areas, and stream profiles; and the location of and approximate yield of fresh water for construction and operation activities.

4-03 TIDAL POWER BASIN

a. Tributary Areas. The area tributary to each pool of the proposed tidal power project is indicated on plate 4-2. The total area includes the watersheds of the Magaguadavic, Digdeguash, St. Croix, and Dennys Rivers as well as numerous other small streams. The area also includes some islands and portions of the shore line not directly associated with any stream system. The land surface is composed of gently rolling lowlands with numerous lakes of varying sizes and a few higher hills along the divides between the watersheds. Most of the area is composed of undeveloped and cutover timberlands, and less than one-tenth of the area is utilized for agricultural activities such as dairying, poultry, or blueberry crops. The major portion of the population is concentrated in a few communities along the coast or in the lower St. Croix valley. The average annual precipitation is about 40 inches and is fairly well distributed throughout the year. The average annual snowfall varies from about 70 inches along the coastal area to almost 100 inches in the inland portions. Average monthly temperatures vary from between 60° and 67° F. in July and August to between 10° and 20° F. in January and February. Extreme temperatures range from a high of 102° F. to a low of 41° F. below zero. The average annual temperature is about 41° F. The daily maximum and minimum temperatures during December, January, and February for the period from December 1935 through January 1951 are shown graphically on plate 4-3 together with the monthly maximum and average measurements of wind velocity. Sea water temperatures, measured during the period December 1935 through March 1936 for various depths of water near St. Andrews, New Brunswick are shown graphically on plate 4-4.

b. Fresh Water Inflow. The St. Croix River is the largest stream emptying into the tidal project pools, draining about 1420 square miles or about 54 percent of the area tributary to the upper pool. The U. S. Geological Survey publishes records of 4 stations on this stream as listed in table 4-1. The annual runoff for the station at Baileyville as summarized in table 4-2 averages 22.39 inches or 1.65 cubic feet per second per square mile. The Water Resources Branch of the Department of Northern Affairs and National Resources of Canada publishes the runoff records of the Magaguadavic River, a tributary of some 710 square miles discharging into the upper pool at St. George. The average annual runoff of the Magaguadavic River as measured at Elmcroft is 2.07 cubic feet per second per square mile. Since the records at this station are available only for intermittent periods since 1916, they were not used in determining the fresh water inflow into the tidal project pool areas. Fresh water inflow into the tidal power pool was estimated by using the St. Croix River records.

c. Area-Capacity Curves. Area-capacity curves for the upper and lower pools of the tidal power project have been prepared on the basis of the most recent field data available and are shown on plates 4-5, 4-6 and 4-7. The following data were examined and where applicable were used in the computation of the area-capacity curves:

(1) Topographic sheets prepared in 1927-28 by Dexter P. Cooper, Inc., to scale 1 inch = 400 feet and 1 inch = 1000 feet. Sheets were compiled from field surveys, U. S. Coast and Geodetic Survey Chart No. 801, and quadrangle maps.

(2) Field books of Cobscook Bay surveys prepared by Dexter P. Cooper, Inc., 1926-27.

(3) Two sets of aerial photographs taken by U. S. Coast and Geodetic Survey specifically for the tidal power survey, scale 1:10,000, flown at approximately the time of mean low tide and half tide on 30 September 1956. Photographs were made on infrared film to produce a sharp contrast between land and water areas.

(4) Topographic maps, scale 1:10,000, compiled by the U. S. Coast and Geodetic Survey from aerial photographs taken in May 1946.

(5) Copies of above maps with mean low tide and half tide lines traced by U. S. Coast and Geodetic Survey from 30 September 1956 photographs. Deviations in feet between mean low water and tide lines were indicated at intervals along the tide lines.

(6) Copies of the original hydrographic sheets, scale 1:10,000, compiled by the U. S. Coast and Geodetic Survey from field surveys of 1887-90 and used as basis for their chart No. 801.

(7) Topographic sheets prepared by the Aero Survey Corporation in November 1956, to scale 1 inch = 400 feet.

(8) Topographic quadrangle sheets, scale 1:50,000, by the Department of Mines and Technical Surveys, Ottawa, Ontario, Canada.

The processing of the data on shore topography was started by comparing Cooper's notes with his maps. His maps had been produced from sounding notes, and the datum was mean sea level. Mean high tide lines and general topography on the 1946 U. S. Coast and Geodetic Survey sheets were then compared with the 1887-90 sheets and with Cooper's sheets by overlaying. The mean high tide lines agreed fairly well on all sources, although the exact locations and extent of some of the smaller islands differed. The locations and outlines of these islands, as shown on 1946 survey sheets, were verified by the 30 September 1956 aerial photographs. Contours above mean high tide as shown on 1946 U. S. Coast and Geodetic Survey sheets were adopted. The low tide and mean tide lines from the 1956 U. S. Coast and Geodetic Survey infrared photographs were correlated with mean sea level. Contours below high tide were then drawn by comparing low and half tide lines with Cooper's topographic sheets. Areas were determined by planimetering the area bounded by each contour on each sheet. From the area data derived in this manner were computed elevation-area curves for the upper and lower pools for a number of possible pool arrangements. From these were computed the elevation-capacity curves which were used in the power computations. Shown on plate 4-5 are the area-capacity curves for the two-pool tidal project layout for which specific designs and cost estimates have been developed. Plates 4-6 and 4-7 show the area-capacity curves for the other tidal project layouts considered in Appendix 5, "Selection of Plan of Development".

4-04 PUMPED-STORAGE BASINS

a. Digdeguash River.

(1) Tributary Area. The Digdeguash basin, a 176-square-mile drainage area, forms a part of the area in southern New Brunswick draining into Passamaquoddy Bay, the upper pool of the proposed tidal project as shown on plate 4-1. The basin is long and narrow and is surrounded by rolling hills. A map of the drainage area, reproduced from the latest available Canadian quadrangle maps, together with a profile of the river are shown on plate 4-8, which also shows the location of the proposed pumped-storage plant near the mouth of the stream. Most of the area is undeveloped timberland with scattered farms and pasturelands. The average annual precipitation is about 42 inches at McAdam, New Brunswick, at the upstream end of the drainage area. The monthly average precipitation ranges from about 3 to 4 inches with the maximum occurring during the late summer or fall months. Temperature extremes range from about 28° F. below zero in winter to around 90° F. in summer.

(2) Stream Flow Records. There are no stream flow records available for the Digdeguash River, and for this reason stream flow was estimated from records for other basins in the vicinity. The Machias River basin at Whitneyville, Maine with a drainage area of 457 square miles and flow records for the period 1906-21, and 1930-54, and located about 40 miles southwest of the Digdeguash River, was selected as the most suitable on the basis of a study of the basin characteristics and flow

records of the streams in the vicinity. Monthly and annual flows were estimated on the basis of drainage area proportion. The Machias River annual runoff and the computed Digdeguash annual runoff are summarized in table 4-3. The computed average annual runoff was estimated to be 265,000 acre-feet (364 c.f.s. average). The computed maximum annual runoff is 382,000 acre-feet and the minimum is 159,000 acre-feet. Computed peak daily discharges during each construction season, June through November, for the period of record are summarized in table 4-4. The computed daily discharges for the flood of September 1954 (estimated to have a 10 percent chance of occurring during any construction season) are tabulated in table 4-5. The daily discharges for the three maximum floods of record (1920, 1936, and 1954) are given in tables 4-6 through 4-8. The peak flow rates have been increased by 10 percent to account for the smaller size of the Digdeguash basin.

(3) Area-Capacity Curves. Area-capacity curves for the reservoir site were determined from maps to a scale of 1 inch =400 feet prepared by the Aero Service Corporation in November 1956. These curves, shown on plate 4-9, indicate the total storage at el. 180 ft. m.s.l. to be 307,000 acre-feet. Since the primary function of the reservoir would be to store energy from the tidal power project, the storage capacity of the reservoir was also computed in terms of energy expressed as kilowatt-hours. The curve of the stored energy above el. 5.0 ft. m.s.l. shown on plate 4-9 was computed on the basis of salt water (64.0 pounds per cubic foot); the storage volumes between succeeding 10-foot increments of elevation, and the head from el. 5.0 to the center of the 10-foot increment being considered.

(4) Stage-Discharge. The flow to and from the pumped-storage plant would pass through the restricted natural passage connecting Digdeguash River to Passamaquoddy Bay. The effect of the restricted passage on the operation of the plant was evaluated by computing water surface profiles for various rates of uniform flow from and to the powerhouse for three arbitrary elevations in Passamaquoddy Bay. The profiles were computed by standard backwater methods using channel cross sections obtained from a survey made in August 1957 by the Eastport Field Office of the U. S. Corps of Engineers. The stage-discharge curves determined by these water surface profiles are shown on plate 4-10.

(5) Unit Hydrograph. The pumped-storage plant being considered at the mouth of Digdeguash River would include a rockfill dam with a spillway as developed in Appendix 11, "Auxiliary Pumped-Storage Developments". The design of the spillway and dam involves consideration of a spillway design flood which, in turn, involves consideration of a unit hydrograph. Because the stream has not been gaged, it was necessary to develop the unit hydrograph by a synthetic method. The method used was based on a paper entitled "Unit-Hydrograph Lag and Peak Flow Related to Basin Characteristics" by Arnold B. Taylor and Harry E. Schwarz published in the Transactions, American Geophysical Union, Volume 33, Number 2, April 1952. The paper presents the results of studies correlating peak flow values and runoff relations to the basin characteristics. Twenty basins in the Northern and Middle Atlantic States ranging from 20 to 1,600 square miles in size were included in the study. The pertinent characteristics of the Digdeguash River basin, as determined from the latest available topographic sheets, were found to fall within the range of basin characteristics presented in the above paper. The synthetic unit hydrograph developed for the Digdeguash River basin on this basis is shown on plate 4-11.

b. Calais Reservoir.

(1) Tributary Area. The Calais reservoir would drain a small oval-shaped area of 19.3 square miles which is surrounded by hills and ridges. The total area would be about twice as large as the reservoir itself, and the portion of the drainage area which would not be occupied by the reservoir is undeveloped timberland. The drainage and reservoir areas are shown on plate 4-12. The climate in the Calais reservoir area is similar to that of the Digdeguash River basin. Temperatures range from about 95° F. in summer to about 23° F. below zero in winter. The average monthly precipitation ranges from about 2.5 to 4.5 inches with a total annual average of about 43 inches.

(2) Stream Flow. Streams in the drainage area have not been gaged and for this reason flow data was estimated from the runoff data for the Machias River at Whitneyville, Maine, 30 miles southwest of the Calais reservoir site, which was also used for the Digdeguash basin studies. Average runoff in c.f.s. for each year from the Calais reservoir drainage area, computed

on the basis of drainage area proportion, is given in table 4-9. This estimate was checked by comparing the runoff in inches over the drainage area with the rainfall for the same years at the U. S. Weather Bureau Station at Woodland, Maine, 32 miles north of Whitneyville on the Machias basin, and with 39 years of record. The land and reservoir losses so computed and given in table 4-9 closely approximated losses computed for other streams in the region. This indicated that the Machias River was representative of the streams in the region and suitable for the area-proportion method for estimating the runoff from other representative areas.

(3) Area-Capacity Curves. Area-capacity curves of the reservoir site were determined from the latest available U. S. Geological Survey quadrangle sheets, scale 1:25,000. These curves are shown on plate 4-13. Also shown is a curve of the stored energy expressed as kilowatt-hours. As for the Digdeguash River, the energy storage value represents the net output of the pumping cycle, or the input of the generating cycle.

(4) Stage-Discharge. The lower bay of the combination powerhouse-pump station would be conjoined to the St. Croix River near its mouth. At this site, the state would be the same as the stage in the upper pool of the tidal power project, Passamaquoddy Bay.

4-05 SAINT JOHN RIVER BASIN

a. General. As developed in detail in Appendix 12, "Auxiliary River Hydro Developments", three sites on the upper Saint John River were considered for the river power auxiliary to the proposed tidal power plant. Of these the Rankin Rapids site was selected for developing a specific design and cost estimate. The other possibilities studied included Big Rapids site, and a combination of the Big Rapids and Lincoln School sites.

b. Climate. The upper Saint John River basin (plate 4-1) has a humid climate with short, cool summers and long, cold winters. Extreme temperatures range from 40° F. below zero in winter to over 100° F. in summer. The average annual precipitation for the upper basin is about 38 inches and ranges from less than 31 to over 42 inches. The average monthly precipitation ranges from about 2 to 5 inches with the annual monthly maximum occurring during the summer months. Winter precipitation is practically all in the form of snow with an annual average of about 100 inches. The snow accumulated by the time of the spring break-up may be equivalent to as much as six inches of water and is an important source of stored water that can be depended on each spring.

c. Tributary Areas. The headwater areas of the Saint John River basin include numerous large lakes and extensive swampy areas in the valleys between rolling hills. A drainage area map is shown on plate 4-14, and a profile of the river is shown on plate 4-15. Drainage areas are tabulated in table 4-10.

d. Stream Flow Records. Stream flow records from the following U. S. Geological Survey gaging stations were used in the studies for the auxiliary river hydro developments; the Saint John River at Dickey, and below Fish River at Fort Kent, the Allegash River near Allagash, and Fish River near Fort Kent. These and other stations located in the basin are listed on table 4-11 and indicated on plate 4-14. All stations are equipped with automatic water stage recorders and their stage discharge relationships are rated as good to excellent, except under ice conditions, which are fair. The monthly records are tabulated on tables 4-12 to 4-15 for the stations used in this study.

(1) Rankin Rapids. Discharge rates were determined from U. S. Geological Survey records by drainage area proportion utilizing the records of the Saint John River at Fort Kent and Fish River near Fort Kent. The drainage area for Rankin Rapids is 4,060 square miles, and at Fort Kent, located about 20 miles downstream, it is 5,690 square miles, including the Fish River drainage area of 871 square miles as shown on plate 4-14. The drainage area and discharge of the Fish River were deducted from those on the Saint John River at Fort Kent and the ratio of the discharge at Rankin Rapids was assumed to be in the same ratio as the drainage areas. The monthly discharges for the period October 1929 through September 1954 computed in this manner for Rankin Rapids are given in table 4-16. The peak daily discharges for each construction season, June through November, computed in the same manner are given in table 4-17. Daily discharge rates for the flood of November 1943 (1 in 10-year construction season) are given in table 4-18. Daily discharges during the three maximum floods of record, May 1933, May 1939, and May 1947 are tabulated in tables 4-19 to 4-21. Table 4-22 lists observed stages and computed flow rates at the dam site.

(2) Big Rapids. Discharge rates for the site were computed from U. S. Geological Survey records by drainage area proportion utilizing the measurements on the Saint John River at Fort Kent and Dickey, Fish River near Fort Kent, and Allagash River near Allagash. Big Rapids is about 3 miles upstream of Dickey and 15 miles upstream of Rankin Rapids, and has a drainage area of 2,419 square miles as shown on plate 4-12. The discharge records at Dickey (2,700 square miles) were reduced in proportion to the drainage area at Big Rapids. Since only eight years of record at Dickey were available, these computations were supplemented by using the records on the Saint John River at Fort Kent, the Fish River, and the Allagash River which are continuous for from 23 to 28 years. The computed monthly discharges at Big Rapids for the period October 1931 through September 1954 are given in table 4-23. Observed stages and computed discharge rates at the dam site are given in table 4-24.

(3) Lincoln School. Stream flow was assumed to be controlled at Big Rapids, about 17 miles upstream of Lincoln School. The total drainage area for Lincoln School and Big Rapids is about the same as for Rankin Rapids. Therefore, the uncontrolled stream flow at Lincoln School was assumed to be the difference between the flow at Rankin and the flow at Big Rapids. The monthly uncontrolled flows at Lincoln School for the period October 1931 through September 1954 computed on this basis are given in table 4-25. Listed in table 4-26 are the observed stages and computed discharges at the dam site.

e. Area-Capacity Curves. Area-capacity curves were prepared for each reservoir site from the latest available U.S. Geological Survey quadrangle sheets, scale 1:62,500. The reservoir shore lines with full pools are as shown on plates 4-16 and 4-17. The area-capacity curves for each of the reservoir sites are shown on plate 4-18.

f. Stage-Discharge.

(1) Rankin Rapids. A stage-discharge relationship was established from water surface elevations observed at the site in 1951-1952 and listed in table 4-22. Corresponding stream flow rates were determined by drainage area proportion from the U. S. Geological Survey daily flow records for Saint John River at Fort Kent (below Fish River), located about 20 miles downstream from Rankin Rapids, and for Fish River near Fort Kent. The computed tailwater rating curve is shown on plate 4-19.

(2) Big Rapids. The shape of the tailwater curve at the Big Rapids site, shown on plate 4-19, was based on the stage-discharge relationship established at Dickey, about three miles downstream, by the U. S. Geological Survey. Stream flow rates were reduced by drainage area proportions from the records at the Dickey station as given in table 4-12. The elevation of the curve was based on the water surface elevations observed at the site in 1957 and the computed corresponding discharges, both listed in table 4-24.

(3) Lincoln School. Stage-discharge relationship was derived from water surface elevations observed at the site in 1957 and given in table 4-26. The shape of the curve was assumed the same as at the Rankin Rapids site, located about two miles upstream with no correction for discharge. The computed tailwater rating curve is shown on plate 4-19.

g. Unit Hydrographs. As for the Digdeguash pumped-storage site, unit hydrographs were needed to develop the spillway design flood. In deriving the unit hydrographs, natural hydrographs for the Saint John River at Dickey and Allagash River at Allagash were plotted for storms occurring during period of record of the gage at Dickey. Twelve-hour unit hydrographs were developed for the upper and lower portions of the Saint John and Allagash basins for the October 1946 and November 1950 storms. The unit hydrographs having the maximum ordinates were selected for each area and are shown on plate 4-20. The storms from which the unit hydrographs were derived were relatively minor, therefore the maximum discharge ordinates were increased by about 25 percent in order that the unit hydrographs would be more representative of the runoff from the large design storm rainfall.

4-06 FRESH WATER SUPPLY

a. Tidal Power Projects. An adequate supply of fresh water, suitable for making concrete, would be required near the site of each major concrete structure. A permanent supply of fresh water would also be required at each powerhouse for operation and maintenance. Examination of available topographic maps indicates that satisfactory water supplies are available in the vicinity of nearly every structure. The locations of the more favorable water supply sites are shown on plate 4-21 and the sizes of the drainage areas are summarized in table 4-27.

b. Pumped-Storage Basin. A suitable supply of fresh water for construction and operation at each site would be available from local streams and nearby lakes. The operation period would be more critical than the construction period because the pumped-storage plants would utilize salt water and thus make the pumped-storage reservoir unsuitable for use during the operation stage. The locations of the most favorable fresh water supply sites for operation are shown on plate 4-21 and the sizes of the drainage areas are summarized in table 4-27.

c. Saint John River Basin. An ample supply of fresh water for construction and operation is available at each site from the Saint John River.

APPENDIX 4

TABLES

TABLE 4-1

U. S. G. S. STREAM FLOW RECORDS
ST. CROIX RIVER BASIN

Location of gaging station	Drainage area (sq. mi.)	Period of record	Discharge c.f.s. Mean[1]	Maximum[2]	Minimum[3]
ST. CROIX RIVER					
Vanceboro, Me.	435	1928-54	680	(Apr.23,1954) 4,470	(Oct.-Nov.1936) 1.9
Baileyville, Me.	1,320	1919-54	2,181	(May 1,1923) 23,300	(July 20,1924) 100
Spragues Falls near Baring, Me.	1,350	1902-05	2,180	(Apr.19,1904) 14,750	(Oct.10,1904) 525
GRAND LAKE STREAM					
Grand Lake Stream, Me.	224	1928-54	343	(June 12,1952) 2,840	(Dec.3-6,11,1945) 5

1 For period of record through September 30, 1954.
2 Instantaneous
3 Daily

TABLE 4-2

ST. CROIX RIVER NEAR BAILEYVILLE, MAINE
ANNUAL RUNOFF IN INCHES
(Drainage Area 1,320 square miles)

Water year ending Sept. 30	Mean runoff[1] in c.f.s.	Runoff in inches per year	Water year ending Sept. 30	Mean runoff[1] in c.f.s.	Runoff in inches per year
1920	2,540	26.11	1938	1,885	19.35
1921	2,120	21.78	1939	2,407	24.71
1922	1,460	14.98	1940	2,053	21.03
1923	2,230	22.89	1941	1,843	18.91
1924	1,740	17.85	1942	1,915	19.63
1925	1,480	15.19	1943	1,706	17.50
1926	2,700	27.70	1944	1,956	20.04
1927	2,310	23.72	1945	2,782	28.60
1928	3.050	31.28	1946	2,171	22.28
1929	2,200	22.58	1947	2,198	22.57
1930	1,860	19.09	1948	1,589	16.30
1931 [2]	1,410	14.46	1949	1,700	17.44
1932	2,110	21.63	1950	1,929	19.79
1933	2,190	22.47	1951 [3]	3,253	33.38
1934	2,388	24.51	1952	2,720	27.92
1935	2,257	23.13	1953	2,171	22.28
1936	2,858	29.38	1954	2,982	30.60
1937	2,200	22.58	Mean	2,182	22.39

1 U.S.G.S. Records.
2 Minimum mean annual runoff.
3 Maximum mean annual runoff.

TABLE 4-3

DIGDEGUASH RIVER, ANNUAL RUNOFF
(Drainage Area 176 square miles)

Water year ending Sept. 30	Mean runoff in c.f.s.		Water year ending Sept. 30	Mean runoff in c.f.s	
	Machias River at Whitney-ville[1]	Digde-guash River at dam site[2]		Machias River at Whitney-ville[1]	Digde-guash River at dam site[2]
1906	1,000	385	1936	995	383
1907	871	335	1937	888	342
1908	1,090	420	1938	943	363
1909	1,200	462	1939	920	354
1910	1,060	408	1940	860	331
1911	578	223	1941 3	566	218
1912	879	339	1942	860	331
1913	1,160	447	1943	817	315
1914	1,020	393	1944	853	329
1915	766	295	1945	1,108	427
1916	890	343	1946	832	320
1917 4	1,360	524	1947	912	351
1918	1,090	420	1948	581	224
1919	1,260	485	1949	695	268
1920	1,200	462	1950	765	295
1921	853	329	1951	1,331	512
1930	824	317	1952	1,176	453
1931	653	251	1953	919	354
1932	802	309	1954	1,348	519
1933	859	331	Mean	946	364
1934	1,041	401			
1935	949	365			

1 U. S. G. S. records.
2 Based on Machias River at Whitneyville.
3 Minimum annual runoff.
4 Maximum annual runoff.

TABLE 4-4

DIGDEGUASH RIVER
CONSTRUCTION SEASON FLOODS, JUNE THROUGH NOVEMBER

Date	Peak average daily discharge in c.f.s.		Order of magnitude
	Machias River at Whitneyville[1]	Digdeguash River at dam site[2]	
Nov. 8, 1907	4,180	1,770	15
Sept. 30, 1909	11,100	4,700	2
Oct. 26, 1912	6,340	2,680	5
July 10, 1915	5,130	2,170	9
June 18, 1917	6,780	2,780	4
Oct. 7, 1918	4,800	2,030	11
Oct. 29, 1932	6,030	2,550	6
Nov. 30, 1935	4,390	1,860	12
Oct. 25, 1937	4,380	1,850	13
June 17, 1942	5,530	2,340	7
Aug. 15, 1943	5,510	2,330	8
Nov. 12, 1944	4,020	1,700	17
June 10, 1947	4,190	1,770	16
Nov. 28, 1950	11,600	4,900	1
Nov. 9, 1951	4,890	2,070	10
Nov. 27, 1953	4,280	1,810	14
Sept. 13, 1954	6,900	2,920	3

1. U. S. G. S. recorded peak average daily discharge, during each construction season, in excess of 4,000 c.f.s., for period of record 1907-21 and 1930-54.
2. Based on Machias River at Whitneyville.

TABLE 4-5

DIGDEGUASH RIVER
SEPTEMBER 1954 FLOOD (1 in 10 YEAR CONSTRUCTION SEASON)

Date 1954	Discharge in c. f. s.	
	Machias River at Whitneyville[1]	Digdeguash River at dam site[2]
Sept. 10	277	120
11	520	220
12	4,800	2,030
13	6,900	2,920
14	5,450	2,310
15	4,420	1,870
16	3,570	1,510
17	2,720	1,150
18	2,080	880
19	1,660	700
20	1,590	670
21	1,610	680
22	1,530	650
23	1,450	610
24	1,310	550
25	1,170	500
26	1,100	470
27	1,040	440
28	979	410
29	914	390

1 U. S. G. S. record.
2. Based on Machias River at Whitneyville.

TABLE 4-6

DIGDEGUASH RIVER, FLOOD OF 1920

Date 1920		Discharge in c.f.s. Machias River at Whitneyville[1]	Digdeguash River at dam site[2]	Date 1920		Discharge in c.f.s. Machias River at Whitneyville[1]	Digdeguash River at dam site[2]
Mar.	22	1,800	760	Apr.	22	4,910	2,080
	23	1,940	820		23	5,020	2,120
	24	2,030	860		24	5,240	2,220
	25	2,480	1,050		25	5,460	2,310
	26	2,950	1,250		26	5,020	2,120
	27	3,750	1,590		27	4,360	1,840
	28	4,800	2,030		28	3,550	1,500
	29	5,350	2,260		29	4,800	2,030
	30	6,780	2,870		30	4,360	1,840
	31	6,890	2,910	May	1	4,150	1,760
Apr.	1	6,340	2,680		2	3,750	1,590
	2	5,900	2,500		3	3,550	1,500
	3	5,020	2,120		4	3,350	1,420
	4	5,020	2,120		5	2,950	1,250
	5	5,020	2,120		6	2,570	1,090
	6	7,000	2,960		7	2,210	940
	7	9,200	3,890		8	1,240	520
	8	8,100	3,430		9	1,700	720
	9	6,780	2,870		10	3,550	1,500
	10	5,680	2,400		11	3,350	1,420
	11	3,950	1,670		12	3,550	1,500
	12	3,750	1,590		13	3,350	1,420
	13	3,950	1,670		14	3,150	1,330
	14	8,100	3,430		15	2,750	1,160
	15	8,980	3,800		16	2,210	940
	16	8,320	3,520		17	2,030	860
	17	7,880	3,330		18	1,780	750
	18	7,000	2,960		19	1,700	720
	19	6,120	2,590		20	1,540	650
	20	5,010	2,120		21	1,310	550
	21	5,240	2,220		22	1,240	520

1. U. S. G. S. records.
2. Based on Machias River at Whitneyville.

TABLE 4-7

DIGDEGUASH RIVER, FLOOD OF 1936

Date 1936	Discharge in c.f.s. Machias River at Whitneyville[1]	Discharge in c.f.s. Digdeguash River at dam site[2]	Date 1936	Discharge in c.f.s. Machias River at Whitneyville[1]	Discharge in c.f.s. Digdeguash River at dam site[2]
Mar. 9	455	190	Apr. 5	3,240	1,370
10	455	190	6	3,150	1,330
11	460	190	7	4,690	1,980
12	1,060	450	8	4,990	2,110
13	5,640	2,390	9	4,490	1,900
14	8,150	3,450	10	3,790	1,600
15	7,600	3,210	11	3,330	1,410
16	6,830	2,890	12	2,970	1,260
17	5,390	2,280	13	2,610	1,100
18	5,790	2,450	14	2,520	1,070
19	6,940	2,940	15	2,610	1,100
20	9,180	3,880	16	2,790	1,180
21	8,700	3,680	17	2,880	1,220
22	7,490	3,170	18	2,790	1,180
23	6,610	2,800	19	2,430	1,030
24	5,590	2,360	20	2,190	930
25	4,390	1,860	21	1,910	810
26	3,240	1,370	22	1,550	660
27	2,700	1,140	23	1,270	540
28	2,970	1,260	24	1,130	480
29	3,420	1,450	25	1,130	480
30	2,880	1,220	26	1,100	470
31	2,270	960	27	1,100	470
Apr. 1	1,950	820	28	1,060	450
2	1,990	840	29	1,060	450
3	2,790	1,180	30	1,200	510
4	3,330	1,410			

1. U. S. G. S. records.
2. Based on Machias River at Whitneyville.

TABLE 4-8

DIGDEGUASH RIVER, FLOOD OF 1954

Date 1954	Discharge in c.f.s.: Machias River at Whitneyville[1]	Discharge in c.f.s.: Digdeguash River at dam site[2]	Date 1954	Discharge in c.f.s.: Machias River at Whitneyville[1]	Discharge in c.f.s.: Digdeguash River at dam site[2]
Apr. 5	898	380	Apr. 28	2,440	1,030
6	866	370	29	2,200	930
7	1,170	500	30	2,000	850
8	2,170	920	May 1	2,260	960
9	2,340	990	2	2,470	1,050
10	1,980	840	3	2,490	1,060
11	1,730	730	4	2,370	1,000
12	2,320	980	5	2,390	1,010
13	2,500	1,060	6	2,940	1,240
14	2,130	900	7	3,650	1,550
15	1,810	770	8	3,550	1,500
16	1,720	730	9	3,200	1,350
17	3,580	1,520	10	3,860	1,630
18	10,000	4,240	11	4,510	1,910
19	11,200	4,740	12	4,430	1,880
20	9,360	3,960	13	3,920	1,660
21	7,640	3,240	14	2,950	1,250
22	6,250	2,640	15	1,870	790
23	5,310	2,250	16	700	300
24	4,720	2,000	17	1,010	430
25	4,220	1,790	18	1,260	530
26	3,640	1,540	19	970	410
27	2,860	1,210	20	1,020	430

1. U. S. G. S. records.
2. Based on Machias River at Whitneyville.

TABLE 4-9

CALAIS RESERVOIR DRAINAGE AREA
ANNUAL RUNOFF IN INCHES

(Drainage Area 19.6 Square Miles)

Water Year ending Sept. 30	Woodland rainfall[1] (inches)	Machias River runoff[2] (inches)	Land area losses (inches)	Calais River water supply net (c.f.s.)
1930	37.71	24.45	13.26	34.8
1931	35.75	19.39	16.36	27.6
1932	38.76	23.88	14.88	34.0
1933	40.31	25.49	14.82	36.2
1934	46.67	30.90	15.77	44.0
1935	41.07	28.17	12.90	40.1
1936	44.98	29.62	15.36	42.1
1937	41.76	26.37	15.39	37.5
1938	46.55	28.02	18.53	39.8
1939	35.68	27.31	8.37	38.8
1940	44.01	25.61	18.40	36.4
1941	34.15	16.80	17.35	23.9
1942	43.93	25.56	18.37	36.3
1943	40.76	24.23	16.53	34.5
1944	43.69	25.44	18.25	36.2
1945	46.65	32.92	13.73	46.8
1946	38.98	24.72	14.26	35.1
1947	40.12	27.08	13.04	38.5
1948	32.68	17.32	15.36	24.6
1949	42.49	20.65	21.84	29.4
1950	42.28	22.75	19.53	32.3
1951	60.51	39.52	20.99	56.2
1952	49.49	35.03	14.46	49.8
1953	48.24	27.29	20.95	38.8
1954	55.34	40.01	15.33	56.9
mean	42.90	26.74	16.16	38.04

1. U. S. Weather Bureau records.
2. U. S. G. S. records.

TABLE 4-10

UPPER SAINT JOHN RIVER BASIN
DRAINAGE AREAS

Location	Drainage area (sq. miles)
Saint John River at Fort Kent (below Fish River)	5,690
Fish River at Fort Kent	871
Allagash River at Allagash	1,250
Saint John River at Lincoln School	4,066
Saint John River at Rankin Rapids	4,060
Saint John River at Dickey	2,700
Saint John River at Big Rapids	2,419

TABLE 4-11

U. S. G. S. STREAM FLOW RECORDS
UPPER SAINT JOHN RIVER BASIN

Location	Drainage area (square miles)	Period of record (Years)	Discharge in c. f. s.		
			Average [1]	Minimum [2]	Maximum [3]
SAINT JOHN RIVER					
Ninemile Bridge[4]	1,290	1950-54	2,309	(Sept. 5,1953) 59	(Apr. 23,1954) 27,800
Dickey	2,700	1910-11;1946-54	4,684	(Sept. 17,1948) 129	(May 9,1947) 68,700
Fort Kent (Below Fish River)[5]	5,690	1926-54	9,596	(Mar. 13-15,1948) 510	(May 5,1933) 121,000
ALLAGASH RIVER					
Allagash	1,250	1910-11;1931-54	1,925	(Mar. 9-15,1948) 91	(May 5,1933) 23,400
SAINT FRANCIS RIVER					
Glazier Lake [4] [5]	496	1951-54	946	(Oct. 6,1953) 88	(Apr. 25,1954) 8,120
FISH RIVER					
Fort Kent	871	1903-08; 1911; 1929-54	1,357	(Oct. 9-10,1950) 46	(Apr. 26,1934) 11,000

1. For period of record through September 30, 1954.
2. Daily.
3. Instantaneous.
4. Not used in studies.
5. International gaging station.

TABLE 4-12

SAINT JOHN RIVER AT DICKEY, MAINE
U. S. G. S. RECORDED MONTHLY AND ANNUAL DISCHARGE, IN C. F. S.
(Drainage area 2,700 square miles)

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1947	5,351	5,009	1,958	1,063	1,949	1,425	7,592	33,430	10,840	6,516	2,211	926	6,565
1948	814	605	703	341	201	742	12,810	18,080	2,510	1,626	1,519	600	3,388
1949	2,827	4,091	2,987	1,178	666	1,065	16,640	8,648	2,717	904	836	1,531	3,671
1950	1,096	2,354	2,601	2,696	1,112	1,179	8,859	9,373	5,878	2,571	1,326	2,212	3,442
1951	2,222	9,368	9,781	1,159	1,393	1,183	22,500	10,050	3,465	5,119	2,240	1,497	5,838
1952	1,880	6,749	3,114	1,440	891	729	11,700	18,170	7,491	937	703	459	4,519
1953	4,011	2,847	1,657	1,246	1,665	2,498	22,140	12,110	1,186	984	265	397	4,246
1954	690	1,728	3,327	775	783	1,798	16,370	15,470	9,540	5,936	5,517	7,655	5,807

TABLE 4-13

SAINT JOHN RIVER BELOW FISH RIVER, AT FORT KENT, MAINE
U. S. G. S. RECORDED MONTHLY AND ANNUAL DISCHARGE, IN C. F. S.
(Drainage area 5,690 square miles)

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1930	5,660	5,710	2,060	3,900	2,070	2,450	15,200	46,700	16,900	9,400	6,660	4,250	10,100
1931	2,940	7,740	3,990	1,860	1,060	1,080	27,200	15,600	11,300	4,360	2,560	5,150	7,060
1932	10,500	11,200	3,480	5,080	2,380	1,750	33,300	28,900	7,610	9,150	7,070	11,400	11,000
1933	7,460	12,300	5,190	2,660	1,560	1,210	16,600	52,900	10,600	6,520	1,980	1,800	10,100
1934	4,683	4,125	2,086	1,316	1,006	1,106	38,370	36,870	9,893	6,335	3,671	2,377	9,341
1935	4,492	9,968	8,423	3,086	1,743	1,742	18,700	28,210	14,920	7,803	2,580	2,293	8,690
1936	2,812	2,213	2,627	1,519	1,235	23,590	25,660	42,130	11,030	3,862	2,263	4,715	10,350
1937	8,083	8,359	3,617	6,379	3,550	3,038	20,810	32,440	10,820	3,535	7,536	4,997	9,459
1938	8,736	13,460	7,016	2,414	2,426	1,940	25,550	28,030	6,724	9,275	7,955	10,800	10,380
1939	7,197	5,212	9,448	3,139	1,376	1,252	6,675	47,830	13,940	8,992	16,400	6,043	10,730
1940	11,620	10,570	7,595	1,926	1,161	1,125	12,570	41,270	19,050	10,420	2,120	1,873	10,150
1941	1,970	10,430	3,916	3,892	1,893	1,396	30,150	16,950	7,589	9,604	3,280	7,518	8,205
1942	10,310	14,310	4,638	2,909	1,528	2,101	20,730	42,290	13,440	4,541	2,155	1,190	10,060
1943	2,901	3,398	1,483	900	688	1,023	5,141	50,550	15,860	6,422	4,781	2,832	8,071
1944	8,960	17,210	2,816	1,185	790	669	3,298	29,960	6,485	4,702	3,465	3,564	6,952
1945	10,000	5,083	2,480	3,397	2,138	4,805	45,570	22,610	11,370	10,790	3,479	5,858	10,630
1946	8,935	5,299	2,886	2,128	1,810	4,510	22,050	40,990	7,132	3,918	3,698	2,053	8,838
1947	6,952	7,432	4,236	2,149	3,941	3,500	14,870	64,120	21,800	10,130	5,813	2,127	12,330
1948	1,578	1,367	1,411	871	562	1,294	21,730	35,490	7,644	4,654	3,469	1,645	6,830
1949	5,375	8,098	7,039	3,016	1,958	2,631	28,820	19,390	8,438	3,813	1,924	2,909	7,785
1950	2,064	4,468	4,621	5,539	2,914	2,795	17,940	20,920	11,340	5,770	2,694	3,381	7,046
1951	3,247	13,840	22,900	4,235	4,230	3,454	41,820	21,550	9,081	10,930	4,684	3,276	11,940
1952	3,240	11,720	7,233	3,608	2,309	2,052	20,770	38,300	16,730	3,852	2,177	1,196	9,436
1953	5,621	4,812	3,552	2,611	3,497	5,171	45,240	27,200	5,374	2,926	1,038	1,105	9,002
1954	1,408	3,356	7,536	2,830	1,971	4,345	29,920	32,550	18,020	14,770	15,520	14,700	12,280

TABLE 4-14

ALLAGASH RIVER NEAR ALLAGASH, MAINE
U. S. G. S. RECORDED MONTHLY AND ANNUAL DISCHARGE, IN C. F. S.
(Drainage area, 1,250 square miles)

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1932	2,300	2,380	699	779	549	455	6,740	5,980	1,790	1,900	1,220	2,860	2,300
1933	2,240	2,670	1,360	586	442	326	3,440	9,620	2,760	1,530	434	356	2,160
1934	1,470	978	619	281	223	241	6,952	7,136	2,721	2,011	765	681	2,012
1935	1,068	1,941	1,936	729	444	356	3,689	5,273	3,758	1,765	527	370	1,836
1936	998	455	505	222	186	3,560	5,005	8,805	2,787	914	323	566	2,035
1937	1,462	1,592	952	1,091	790	819	3,803	7,766	2,688	884	1,374	812	2,010
1938	1,119	2,119	1,339	578	351	557	4,553	6,162	1,603	2,158	1,166	2,482	2,022
1939	1,759	1,060	1,818	728	280	288	1,315	9,347	3,502	2,583	3,077	1,159	2,265
1940	2,201	2,459	1,450	483	252	267	3,210	6,949	4,331	2,991	614	489	2,146
1941	242	1,311	635	1,175	477	397	4,761	3,440	2,091	1,381	1,324	966	1,517
1942	1,236	2,447	1,111	465	357	455	3,836	8,067	2,125	1,398	690	365	1,888
1943	538	385	271	198	145	191	716	8,902	3,462	2,114	1,181	734	1,584
1944	1,557	2,922	596	243	220	196	623	4,528	1,333	1,054	815	1,515	1,303
1945	2,099	982	608	561	429	751	8,699	4,863	2,354	2,981	1,043	1,083	2,207
1946	1,730	996	523	461	403	594	3,102	8,987	1,724	1,910	921	576	1,842
1947	741	1,113	924	446	850	852	3,142	11,700	4,544	1,458	1,858	609	2,365
1948	315	279	286	192	119	241	2,545	6,835	1,665	1,716	931	358	1,297
1949	717	1,595	1,586	708	524	826	5,443	3,901	2,177	822	320	307	1,577
1950	273	637	776	1,210	710	662	3,481	4,251	2,147	1,353	726	770	1,418
1951	582	2,344	4,549	1,224	1,144	791	6,302	4,828	2,492	1,625	764	611	2,273
1952	462	1,921	1,415	648	522	424	3,352	8,205	3,580	1,186	355	247	1,863
1953	579	706	599	496	594	983	7,985	5,823	1,409	668	241	301	1,698
1954	309	729	1,741	884	502	906	5,112	6,958	4,163	4,053	3,353	3,133	2,664

TABLE 4-15

FISH RIVER NEAR FORT KENT, MAINE
U. S. G. S. RECORDED MONTHLY AND ANNUAL DISCHARGE, IN C. F. S.
(Drainage area, 871 square miles)

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1930	351	498	285	412	338	480	2,930	6,040	2,790	1,540	886	685	1,440
1931	490	476	471	305	239	230	3,160	2,520	1,470	541	290	370	880
1932	1,310	1,770	671	628	437	425	4,620	4,250	1,210	748	1,170	1,370	1,550
1933	1,230	1,840	1,290	488	315	305	2,250	6,080	1,890	1,190	307	250	1,460
1934	471	692	597	295	161	250	4,400	5,589	1,446	933	526	269	1,307
1935	510	1,212	1,188	626	367	284	2,191	4,851	1,979	1,131	315	230	1,246
1936	151	187	324	289	258	3,104	4,719	5,346	2,122	700	333	480	1,504
1937	708	1,393	769	733	621	744	2,540	4,440	1,495	429	640	263	1,234
1938	431	1,913	1,502	685	510	518	2,843	4,228	1,507	1,872	650	757	1,456
1939	766	796	1,434	777	312	320	1,134	6,242	1,775	1,334	2,825	1,139	1,585
1940	1,549	2,184	1,755	544	325	287	2,204	5,756	2,338	1,907	420	213	1,629
1941	205	1,162	822	729	413	322	3,367	3,857	1,138	2,555	569	738	1,328
1942	1,410	2,045	929	514	392	464	2,395	5,683	2,156	943	298	128	1,453
1943	146	258	232	157	139	245	554	6,251	2,633	825	561	447	1,045
1944	741	2,563	742	210	164	107	390	3,685	1,089	603	345	209	906
1945	1,030	1,388	725	679	531	649	5,974	3,722	1,625	1,691	650	518	1,599
1946	1,000	866	594	441	336	525	2,550	6,635	1,647	672	345	273	1,332
1947	188	557	821	296	626	700	2,176	8,036	3,498	950	417	219	1,547
1948	134	278	303	203	119	175	2,691	5,051	1,928	585	528	281	1,025
1949	748	1,237	1,259	730	399	384	3,794	3,404	1,508	947	341	197	1,248
1950	190	688	800	1,134	703	551	2,598	3,992	1,321	679	275	171	1,094
1951	90	938	4,688	1,172	982	730	5,958	3,261	1,498	1,971	851	601	1,899
1952	410	1,467	1,479	962	618	366	2,735	6,475	2,776	658	230	122	1,527
1953	267	463	623	425	686	838	7,495	4,897	1,145	356	160	100	1,452
1954	124	427	1,616	687	399	813	3,665	5,423	2,466	3,075	3,571	1,940	2,029

TABLE 4-16

SAINT JOHN RIVER AT RANKIN RAPIDS, MAINE
COMPUTED MONTHLY AND ANNUAL DISCHARGE, IN C. F. S.
(Drainage area 4,060 square miles)

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1930	4,470	4,388	1,495	2,937	1,374	1,659	10,331	34,236	11,881	6,618	4,862	3,002	7,271
1931	2,063	6,116	2,963	1,309	691	716	20,242	11,013	8,277	3,216	1,911	4,025	5,212
1932	7,738	7,940	2,365	3,749	1,636	1,116	24,149	20,755	5,389	7,074	4,968	8,445	7,944
1933	5,246	8,807	3,284	1,829	1,048	762	12,121	39,422	7,334	4,488	1,409	1,305	7,256
1934	3,546	2,891	1,254	860	711	721	28,603	26,339	7,112	4,548	2,648	1,775	6,751
1935	3,353	7,373	6,092	2,071	1,159	1,228	13,901	19,668	10,896	5,618	1,907	1,732	6,249
1936	2,247	1,706	1,940	1,036	823	17,249	17,632	30,972	7,501	2,662	1,625	3,566	7,413
1937	6,210	5,865	2,398	4,754	2,466	1,932	15,383	23,576	7,852	2,615	5,806	3,986	6,904
1938	6,993	9,723	4,643	1,456	1,613	1,197	19,119	20,041	4,393	6,233	6,151	8,456	7,502
1939	5,415	3,718	6,748	1,990	895	785	4,660	35,010	10,240	6,450	11,430	4,130	7,623
1940	8,480	7,060	4,920	1,160	705	710	8,700	29,900	14,070	7,170	1,430	1,400	7,142
1941	1,490	7,800	2,605	2,665	1,245	905	22,550	11,025	5,430	5,935	2,285	5,710	5,804
1942	7,495	10,325	3,120	2,015	955	1,380	15,440	30,825	9,500	3,030	1,565	895	7,212
1943	2,320	2,645	1,055	625	460	655	3,860	37,300	11,140	4,710	3,555	2,010	5,861
1944	6,920	12,335	1,750	820	530	475	2,450	22,125	4,545	3,450	2,625	2,825	5,071
1945	7,555	3,120	1,480	2,290	1,350	3,500	33,345	15,905	8,205	7,660	2,380	4,495	7,607
1946	6,680	3,730	1,930	1,420	1,240	3,355	16,420	28,925	4,620	2,735	2,825	1,500	6,282
1947	5,695	5,790	2,875	1,560	2,790	2,360	10,688	47,225	15,410	7,730	4,545	1,605	9,023
1948	1,215	910	935	564	375	940	16,030	25,630	4,810	3,425	2,475	1,150	4,872
1949	3,895	5,775	4,865	1,925	1,313	1,892	21,072	13,460	5,835	2,413	1,333	2,284	5,505
1950	1,578	3,183	3,217	3,709	1,862	1,889	12,918	14,253	8,436	4,286	2,037	2,703	5,006
1951	2,658	10,863	15,335	2,579	2,735	2,294	30,196	15,399	6,385	7,543	3,227	2,252	8,456
1952	2,383	8,633	4,845	2,228	1,424	1,420	15,185	26,839	11,749	2,689	1,639	904	6,662
1953	4,508	3,662	2,466	1,840	2,367	3,648	31,781	18,779	3,561	2,164	739	846	6,363
1954	1,081	2,466	4,985	1,804	1,324	2,974	22,107	22,841	13,096	9,847	10,061	10,774	8,613

TABLE 4-17

SAINT JOHN RIVER AT RANKIN RAPIDS
CONSTRUCTION SEASON FLOODS, JUNE THROUGH NOVEMBER

Date	Peak average daily discharge, c.f.s.[1] At Fort Kent below Fish River[2]	Fish River at Fort Kent[2]	Net	Rankin Rapids	Order of magnitude
June 24, 1930	19,200	2,550	16,650	14,000	18
Oct. 19, 1931	25,100	1,690	23,410	19,700	13
Sept. 19, 1932	53,600	2,030	51,570	43,400	1
July 2, 1933	19,600	1,860	17,740	14,900	17
June 14, 1934	16,500	1,430	15,070	12,700	21
June 17, 1935	22,000	2,240	19,760	16,600	16
Nov. 5, 1936	17,200	1,760	15,440	13,000	20
Nov. 1, 1937	23,300	1,700	21,600	18,200	14
Sept. 23, 1938	27,900	827	27,073	22,800	11
Aug. 6, 1939	32,500	2,910	29,590	24,900	8
June 21, 1940	36,300	2,160	34,140	28,700	6
Nov. 4, 1941	23,600	2,030	21,570	18,200	15
June 19, 1942	44,400	3,370	41,030	34,500	4
Nov. 11, 1943	42,600	3,550	39,050	32,900	5
Oct. 11, 1944	16,900	778	16,122	13,600	19
July 17, 1945	27,000	2,320	24,680	20,800	12
Oct. 3, 1946	29,800	287	29,513	24,800	9
July 30, 1947	28,900	732	28,168	23,700	10
Nov. 30, 1948	12,700	1,550	11,150	9,390	24
June 28, 1949	15,600	1,680	13,920	11,700	23
Nov. 30, 1950	48,300	4,170	44,130	37,200	3
July 8, 1951	34,800	2,590	32,210	27,100	7
Oct. 9, 1952	15,100	299	14,801	12,500	22
Nov. 29, 1953	8,380	1,070	7,310	6,160	25
June 29, 1954	46,900	1,860	45,040	37,900	2

1 Annual spring floods not included.
2 U. S. G. S. records.

TABLE 4-18

SAINT JOHN RIVER AT RANKIN RAPIDS
OCTOBER-NOVEMBER 1943 FLOOD (1 in 10 YEAR CONSTRUCTION SEASON)

Date 1943	Discharge in c. f. s. At Fort Kent below Fish R.[1]	Fish River at Fort Kent[1]	Net	Rankin Rapids
Oct. 16	1,660	361	1,299	1,094
17	2,030	432	1,598	1,345
18	9,050	568	8,482	7,140
19	24,500	681	23,819	20,055
20	25,200	901	24,299	20,460
21	22,300	1,050	21,250	17,890
22	19,500	1,170	18,330	15,435
23	16,100	1,270	14,830	12,485
24	14,000	1,360	12,640	10,645
25	12,700	1,460	11,240	9,465
26	11,400	1,520	9,880	8,320
27	10,800	1,520	9,280	7,815
28	11,600	1,630	9,970	8,395
29	17,500	1,740	15,760	13,270
30	23,100	1,800	21,300	17,935
31	24,000	1,980	22,020	18,540
Nov. 1	21,100	2,040	19,060	16,050
2	17,900	2,040	15,860	13,355
3	16,100	2,040	14,060	11,840
4	17,100	2,180	14,920	12,560
5	18,300	2,180	16,120	13,575
6	17,100	2,240	14,860	12,510
7	17,500	2,440	15,060	12,680
8	21,900	2,720	19,180	16,150
9	25,300	2,950	22,350	18,820
10	40,300	3,400	36,900	31,070
11	42,600	3,550	39,050	32,880
12	35,300	3,620	31,680	26,675
13	28,900	3,620	25,280	21,285
14	24,400	3,550	20,850	17,555
15	20,700	3,480	17,220	14,500
16	17,900	3,180	14,720	12,395
17	15,400	3,100	12,300	10,355
18	13,300	2,950	10,350	8,715
19	12,000	2,800	9,200	7,745
20	11,400	2,650	8,750	7,365
21	10,800	2,580	8,220	6,920
22	10,200	2,440	7,760	6,535
23	9,630	2,300	7,330	6,170
24	8,550	2,180	6,370	5,365
25	7,820	2,040	5,780	4,865
26	7,590	1,920	5,670	4,775

1. U. S. G. S. records.

TABLE 4-19

SAINT JOHN RIVER AT RANKIN RAPIDS
FLOOD OF 1933

Date 1933		Discharge in c. f. s. At Fort Kent below Fish R.[1]	Fish River at Fort Kent[1]	Net	Rankin Rapids
Apr.	11	2,210	750	1,460	1,230
	12	2,280	860	1,420	1,190
	13	2,870	965	1,905	1,600
	14	3,640	1,060	2,580	2,170
	15	4,690	1,330	3,360	2,830
	16	6,330	1,580	4,750	4,000
	17	7,450	1,860	5,590	4,710
	18	10,100	2,280	7,820	6,580
	19	15,400	2,690	12,710	10,700
	20	22,200	3,130	19,070	16,100
	21	27,100	3,680	23,420	19,700
	22	51,000	4,260	46,740	39,400
	23	46,500	4,430	42,070	35,400
	24	43,400	4,600	38,800	32,700
	25	41,500	4,770	36,730	30,900
	26	44,000	4,940	39,060	32,900
	27	41,500	4,940	36,560	30,800
	28	38,500	4,940	33,560	28,200
	29	36,100	4,940	31,160	26,200
	30	36,700	5,110	31,590	26,600
May	1	37,300	5,460	31,840	26,800
	2	51,000	6,310	44,690	37,600
	3	89,300	8,230	81,070	68,300
	4	115,000	9,450	105,550	89,000
	5	117,000	9,630	107,370	90,400
	6	104,000	9,280	94,720	79,800
	7	81,000	8,930	72,070	60,700
	8	76,100	8,400	67,700	57,000
	9	63,100	8,230	54,870	46,200
	10	58,400	7,880	50,520	42,540
	11	55,000	7,530	47,470	40,000
	12	57,000	7,350	49,650	41,800
	13	57,700	7,010	50,690	42,700
	14	57,700	6,830	50,870	42,800
	15	68,600	6,830	61,770	52,000

1. U. S. G. S. records.

TABLE 4-19 (Continued)

SAINT JOHN RIVER AT RANKIN RAPIDS
FLOOD OF 1933

Date 1933		Discharge in c. f. s. At Fort Kent below Fish R.[1]	Fish River at Fort Kent[1]	Net	Rankin Rapids
May	16	62,500	6,660	55,840	47,000
	17	59,100	6,310	52,790	44,400
	18	47,100	5,970	41,130	34,600
	19	45,800	5,800	40,000	33,700
	20	45,800	5,290	40,510	34,100
	21	42,700	5,110	37,590	31,700
	22	37,900	4,770	33,130	27,900
	23	30,900	4,430	26,470	22,300
	24	27,100	4,090	23,010	19,400
	25	22,700	3,760	18,940	15,900
	26	20,500	3,440	17,060	14,400
	27	19,600	3,280	16,320	13,700
	28	20,900	3,280	17,620	14,800
	29	24,600	3,130	21,470	18,100
	30	23,100	2,980	20,120	16,900
	31	21,800	2,980	18,820	15,800
June	1	20,500	2,830	17,670	14,900
	2	19,600	2,690	16,910	14,200
	3	17,600	2,620	14,980	12,600
	4	13,400	2,480	10,920	9,190
	5	12,400	2,340	10,060	8,470
	6	11,800	2,220	9,580	8,070
	7	11,500	2,090	9,410	7,920
	8	9,510	1,970	7,540	6,350
	9	9,710	1,800	6,910	5,820
	10	8,190	1,800	6,390	5,380
	11	11,200	1,740	9,460	7,970
	12	8,970	1,690	7,280	6,130
	13	8,450	1,580	6,870	5,780
	14	8,970	1,480	7,490	6,310
	15	8,970	1,430	7,540	6,350

1. U. S. G. S. records.

TABLE 4-20

SAINT JOHN RIVER AT RANKIN RAPIDS
FLOOD OF 1939

Date 1939		Discharge in c. f. s. At Fort Kent below Fish R.[1]	Fish River at Fort Kent[1]	Net	Rankin Rapids
Apr.	21	2,380	1,880	500	420
	22	4,140	2,390	1,750	1,480
	23	7,140	2,530	4,610	3,880
	24	9,140	2,740	6,400	5,390
	25	13,600	2,670	10,930	9,200
	26	18,900	2,390	16,510	13,900
	27	20,800	2,460	18,340	15,400
	28	26,000	2,670	23,330	19,600
	29	36,300	3,230	33,070	27,800
	30	37,400	3,580	33,820	28,500
May	1	45,200	3,900	41,300	34,800
	2	41,500	4,140	37,360	31,500
	3	42,100	4,460	37,640	31,700
	4	47,700	4,800	42,900	36,100
	5	54,100	5,310	48,790	41,100
	6	58,700	5,650	53,050	44,700
	7	64,700	6,170	58,530	49,300
	8	77,100	7,470	69,630	58,600
	9	93,600	8,800	84,800	71,400
	10	109,000	10,100	98,900	83,300
	11	115,000	10,700	104,300	87,800
	12	105,000	10,300	94,700	79,700
	13	87,000	9,940	77,060	64,900
	14	69,500	9,370	60,130	50,600
	15	56,000	8,990	47,010	39,600
	16	47,100	8,610	38,490	32,400
	17	40,300	8,230	32,070	27,000
	18	36,300	7,470	28,830	24,300
	19	34,600	6,900	27,700	23,300
	20	30,400	6,350	24,050	20,200
	21	26,500	5,820	20,680	17,400
	22	23,700	5,480	18,220	15,300
	23	20,800	4,970	15,830	13,300
	24	18,600	4,630	13,970	11,800
	25	17,500	4,140	13,360	11,200
	26	15,700	3,900	11,800	9,940
	27	13,600	3,580	10,020	8,440
	28	15,000	3,580	11,420	9,620

1. U. S. G. S. records.

TABLE 4-20 (Continued)

SAINT JOHN RIVER AT RANKIN RAPIDS
FLOOD OF 1939

Date 1939	Discharge in c. f. s. At Fort Kent below Fish R. [1]	Fish River at Fort Kent [1]	Net	Rankin Rapids
May 29	19,700	3,500	16,200	13,600
30	27,900	3,200	24,700	20,800
31	28,900	3,050	25,850	21,800
June 1	26,500	3,050	23,450	19,700
2	22,500	2,910	19,590	16,500
3	18,900	2,770	16,130	13,600
4	16,400	2,630	13,770	11,600
5	15,000	2,490	12,510	10,500
6	14,000	2,360	11,640	9,800
7	14,300	2,230	12,070	10,200
8	14,300	2,160	12,140	10,200
9	13,600	2,040	11,560	9,730
10	15,700	1,910	13,790	11,600
11	15,000	1,850	13,150	11,100
12	14,000	1,730	12,270	10,300
13	13,600	1,680	11,920	10,000
14	14,300	1,730	12,570	10,600
15	17,500	1,790	15,710	13,200
16	19,700	1,730	17,970	15,100
17	18,600	1,680	16,920	14,200
18	16,400	1,620	14,780	12,400
19	14,000	1,560	12,440	10,500
20	12,000	1,510	10,490	8,830
21	10,500	1,400	9,100	7,660
22	9,140	1,300	7,840	6,600
23	7,610	1,260	6,350	5,350
24	7,370	1,240	6,130	5,160
25	8,870	1,180	7,690	6,470
26	10,500	1,150	9,350	7,870
27	10,800	1,090	9,710	8,180
28	9,410	1,040	8,370	7,050
29	8,610	992	7,620	6,420
30	9,140	1,170	7,970	6,710

1. U. S. G. S. records.

TABLE 4-21

SAINT JOHN RIVER AT RANKIN RAPIDS
FLOOD OF 1947

Date 1947		Discharge in c. f. s.			
		At Fort Kent below Fish R.[1]	Fish River at Fort Kent [1]	Net	Rankin Rapids
Apr.	1	4,850	670	4,180	3,520
	2	5,040	664	4,376	3,680
	3	5,040	652	4,388	3,680
	4	4,850	658	4,192	3,530
	5	5,230	658	4,572	3,850
	6	5,630	728	4,902	4,120
	7	6,470	932	5,538	4,660
	8	7,140	1,040	6,100	5,130
	9	8,350	1,100	7,250	6,100
	10	10,500	1,200	9,300	7,830
	11	12,400	1,440	10,960	9,230
	12	15,600	2,000	13,600	11,500
	13	19,100	2,180	16,920	14,200
	14	21,500	2,380	19,120	16,100
	15	24,000	2,800	21,200	17,900
	16	25,600	2,660	22,940	19,300
	17	24,400	2,800	21,600	18,200
	18	23,100	2,800	20,300	17,100
	19	21,500	2,800	18,700	15,700
	20	19,900	2,870	17,030	14,300
	21	18,700	2,870	15,830	13,300
	22	17,600	2,800	14,800	12,400
	23	16,600	2,800	13,800	11,600
	24	16,600	3,010	13,590	11,400
	25	17,900	3,240	14,660	12,300
	26	19,100	3,400	15,700	13,200
	27	17,200	3,550	13,650	11,500
	28	16,600	3,550	13,050	11,000
	29	17,200	3,480	13,720	11,600
	30	18,300	3,550	14,750	12,400
May	1	20,700	3,550	17,150	14,450
	2	21,500	3,550	17,950	15,120
	3	21,500	3,630	17,870	15,050
	4	23,900	4,280	19,620	16,520
	5	35,300	5,710	29,590	24,900
	6	59,900	6,940	52,960	44,600
	7	82,900	8,920	73,980	62,300

1. U. S. G. S. records.

TABLE 4-21 (Continued)

SAINT JOHN RIVER AT RANKIN RAPIDS
FLOOD OF 1947

Date 1947	Discharge in c. f. s. At Fort Kent below Fish R.[1]	Fish River at Fort Kent[1]	Net	Rankin Rapids
May 8	107,000	10,600	96,400	81,200
9	111,000	10,800	100,200	84,400
10	94,600	10,200	84,400	71,100
11	79,500	9,840	69,660	58,600
12	71,200	9,660	61,540	51,800
13	77,000	10,000	67,000	56,400
14	85,800	10,200	75,600	63,600
15	79,500	9,840	69,660	58,600
16	69,800	9,470	60,330	50,800
17	63,500	9,100	54,400	45,800
18	62,800	8,920	53,880	45,300
19	64,800	8,920	55,880	47,100
20	69,100	8,920	60,180	50,700
21	76,400	8,920	67,480	56,800
22	79,500	8,920	70,580	59,400
23	80,200	8,730	71,470	60,200
24	74,900	8,360	66,540	56,000
25	66,900	7,990	58,910	49,600
26	59,400	7,810	51,590	43,400
27	60,800	7,620	53,180	44,800
28	55,600	7,440	48,160	40,600
29	49,300	7,250	42,050	35,400
30	43,800	6,700	37,100	31,200
31	39,700	6,340	33,360	28,100
June 1	34,300	5,980	28,320	23,900
2	29,400	5,630	23,770	20,000
3	26,600	5,280	21,320	18,000
4	24,000	4,930	19,070	16,100
5	20,800	4,580	16,220	13,700
6	20,600	4,580	16,020	13,500
7	25,600	4,320	21,280	17,900
8	26,600	4,160	22,440	19,000
9	23,100	3,890	19,210	16,200
10	23,100	3,720	19,380	16,310
11	22,000	3,550	18,450	15,500
12	19,500	3,210	16,290	13,700
13	19,500	3,120	16,380	13,800
14	18,700	2,880	15,820	13,300

1. U. S. G. S. records.

TABLE 4-21 (Continued)

SAINT JOHN RIVER AT RANKIN RAPIDS
FLOOD OF 1947

Date 1947	Discharge in c. f. s. At Fort Kent below Fish R.[1]	Fish River at Fort Kent[1]	Net	Rankin Rapids
June 15	19,900	3,120	16,780	14,100
16	30,400	3,300	27,100	22,800
17	32,800	3,210	29,590	24,900
18	27,500	3,210	24,290	20,500
19	26,600	3,300	23,300	19,600
20	29,800	3,300	26,500	22,300
21	27,500	3,210	24,290	20,500
22	23,100	3,040	20,060	16,900
23	19,100	3,040	16,060	13,500
24	16,600	2,880	13,720	11,600
25	15,200	2,720	12,480	10,500
26	13,900	2,490	11,410	9,610
27	13,000	2,340	10,660	8,980
28	11,400	2,180	9,220	7,760
29	7,140	1,960	5,180	4,360
30	6,250	1,820	4,430	3,730

1. U. S. G. S. records.

TABLE 4-22

SAINT JOHN RIVER AT RANKIN RAPIDS
OBSERVED STAGES AND COMPUTED RATES OF FLOW

Date		Water surface elev.[1] ft. m.s.l.	Discharge in c. f. s.			
			At Fort Kent[2] below Fish R.	Fish R.[2] at Fort Kent	Net	Rankin Rapids
1951						
May	8	552.44	23,600	4,410	19,190	16,158
	9	552.52	22,900	4,190	18,710	15,754
	10	552.39	22,100	3,950	18,150	15,282
	11	552.33	21,200	3,750	17,450	14,693
	12	552.19	19,500	3,510	15,990	13,464
	13	552.14	19,400	3,370	16,030	13,497
	14	552.1	19,000	3,210	15,790	13,295
	15	551.8	17,600	2,960	14,640	12,327
	16	551.6	16,500	2,780	13,720	11,552
	17	551.6	17,100	2,620	14,480	12,192
	18	551.6	16,700	2,470	14,230	11,982
	19	551.3	14,100	2,310	11,790	9,927
	20	551.2	12,200	2,190	10,010	8,428
	21	550.3	11,300	2,070	9,230	7,772
Aug.	20	549.9	6,570	726	5,844	4,920
	29	548.44	4,220	864	3,356	2,825
1952						
June	10	552.99	27,900	3,480	24,420	20,562
Aug.	4	547.36	1,590	287	1,303	1,097
	5	547.41	1,590	291	1,299	1,094
	6	547.56	1,700	287	1,413	1,190
	7	547.91	2,090	276	1,814	1,527
	8	547.78	2,130	272	1,858	1,564
	9	547.63	1,940	261	1,679	1,414
	10	547.54	1,770	253	1,517	1,277
	11	547.45	1,670	246	1,424	1,199
	12	547.43	1,550	235	1,315	1,107
	13	547.41	1,510	225	1,285	1,082

1. U. S. Corps of Engineers temporary gage.
2. U. S. G. S. records.

TABLE 4-23

SAINT JOHN RIVER AT BIG RAPIDS, MAINE
COMPUTED MONTHLY AND ANNUAL DISCHARGE, IN C. F. S.
(Drainage area 2,419 square miles)

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1932	4,682	4,787	1,434	2,557	936	569	14,989	12,721	3,099	4,455	3,227	4,809	4,855
1933	2,588	5,284	1,657	1,070	522	375	7,474	25,660	3,938	2,547	835	817	4,397
1934	1,787	1,647	547	498	420	413	18,642	16,534	3,781	2,184	1,621	942	4,085
1935	1,967	4,677	3,578	1,155	616	751	8,793	12,394	6,146	3,317	1,188	1,173	3,816
1936	1,075	1,077	1,236	701	548	11,786	10,872	19,086	4,059	1,505	1,121	2,583	4,637
1937	4,088	3,679	1,245	3,154	1,443	958	9,970	13,612	4,446	1,490	3,816	2,733	4,220
1938	5,058	6,547	2,845	756	1,087	551	12,541	11,950	2,402	3,509	4,292	5,144	4,724
1939	3,148	2,289	4,245	1,087	530	428	2,889	22,096	5,801	3,329	7,192	2,558	4,633
1940	5,406	3,961	2,988	583	390	381	4,727	19,761	8,385	3,598	703	784	4,306
1941	1,075	5,587	1,696	1,369	661	437	15,316	6,531	2,875	3,922	827	4,085	3,698
1942	5,389	6,783	1,730	1,335	515	796	9,991	19,595	6,350	1,405	753	456	4,592
1943	1,534	1,946	675	368	271	400	2,707	24,451	6,611	2,235	2,044	1,099	3,695
1944	4,618	8,105	994	497	267	240	1,573	15,151	2,766	2,063	1,558	1,128	3,247
1945	4,698	1,841	751	1,489	793	2,367	21,220	9,507	5,038	4,029	1,065	2,938	4,645
1946	4,262	2,354	1,211	826	721	2,377	11,467	17,167	2,493	710	1,639	796	3,831
1947	4,265	4,027	1,680	959	1,670	1,298	6,497	30,587	9,356	5,400	2,314	858	5,742
1948	775	543	559	320	220	602	11,611	16,182	2,708	1,471	1,329	682	3,084
1949	2,736	3,599	2,823	1,048	679	918	13,457	8,230	3,150	1,370	872	1,702	3,382
1950	1,124	2,192	2,102	2,324	992	1,056	8,125	8,612	5,414	2,474	1,129	1,664	3,101
1951	1,787	7,335	9,287	1,167	1,370	1,294	20,573	9,102	3,352	5,095	2,121	1,413	5,325
1952	1,654	5,779	2,953	1,360	777	858	10,188	16,044	7,034	1,294	1,106	566	4,134
1953	3,383	2,545	1,607	1,157	1,527	2,295	20,488	11,155	1,853	1,288	429	469	4,016
1954	665	1,496	2,793	792	708	1,781	14,633	13,652	7,691	4,989	5,776	6,579	5,130

TABLE 4-24

SAINT JOHN RIVER AT BIG RAPIDS
OBSERVED STAGES AND COMPUTED RATES OF FLOW

		Discharge in c. f. s.						
Date	Water surface elev.[1] ft. m.s.l.	At Fort Kent[2] below Fish R.	Fish R.[2] at Fort Kent	Net	Rankin Rapids	Allagash R.[2] at Allagash	Net	Big Rapids
1957								
Oct.								
8	628.5	2,380	193	2,187	1,841	371	1,470	1,266
9	628.6	2,230	193	2,037	1,715	353	1,362	1,173
10	628.7	2,130	193	1,937	1,631	353	1,278	1,100
11	628.6	2,040	186	1,854	1,561	353	1,208	1,040
12	628.5	1,960	176	1,784	1,502	337	1,165	1,003
13	628.4	1,950	170	1,780	1,499	321	1,178	1,014
14	628.3	1,790	163	1,627	1,370	315	1,055	908
15	628.3	1,730	160	1,570	1,322	304	1,018	876
16	628.3	1,680	160	1,520	1,280	299	981	845
17	628.3	1,630	160	1,470	1,238	299	939	808
18	628.3	1,590	160	1,430	1,204	299	905	779
19	628.4	1,640	170	1,470	1,238	331	907	781
21	630.1	3,410	207	3,203	2,697	623	2,074	1,786
22	630.3	4,930	204	4,726	3,979	524	3,455	2,975
23	630.1	4,810	200	4,610	3,882	480	3,402	2,929
24	629.9	4,340	207	4,133	3,480	509	2,971	2,558
25	630.5	4,850	275	4,575	3,852	791	3.061	2,636
26	631.4	7,600	267	7,333	6,174	920	5,254	4,524
27	631.7	9,160	255	8,905	7,498	791	6,707	5,775
28	631.1	7,800	255	7,545	6,353	706	5,647	4,862
29	630.2	6,580	251	6,329	5,329	672	4,657	4,010
30	629.8	5,640	255	5,385	4,534	623	3,911	3,367
31	629.2	5,050	259	4,791	4,034	600	3,434	2,957
Nov.								
1	628.6	4,720	259	4,461	3,756	576	3,180	2,738
3	630.7	4,590	301	4,289	3,611	623	2,988	2,573
4	631.3	6,760	443	6,317	5,319	1,680	3,639	3,133
5	631.1	11,000	514	10,486	8,829	2,440	6,389	5,501
6	631.0	12,500	542	11,958	10,069	2,260	7,809	6,724
7	631.0	11,800	589	11,211	9,440	2,020	7,420	6,389
8	630.8	10,400	638	9,762	8,220	1,860	6,360	5,476

1. U. S. Corps of Engineers temporary gage.
2. U. S. G. S. provisional records

TABLE 4-24 (Continued)

SAINT JOHN RIVER AT BIG RAPIDS
OBSERVED STAGES AND COMPUTED RATES OF FLOW

Date	Water surface elev.[1] ft. m.s.l.	Discharge in c.f.s. At Fort Kent[2] below Fish R.	Fish R.[2] at Fort Kent	Net	Rankin Rapids	Allagash R.[2] at Allagash	Net	Big Rapids
1957								
Nov.								
9	631.0	10,700	889	9,811	8,261	2,380	5,881	5,064
10	631.4	13,700	973	12,727	10,716	3,020	7,696	6,626
12	631.5	13,100	1,090	12,010	10,112	2,220	7,892	6,795
13	631.6	11,400	1,140	10,260	8,639	2,040	6,599	5,682
14	631.7	10,300	1,170	9,130	7,687	1,920	5,767	4,965
15	631.8	9,840	1,280	8,560	7,207	1,930	5,277	4,543
16	631.7	11,100	1,450	9,650	8,125	2,470	5,655	4,869
18	632.1	17,600	1,810	15,790	13,295	2,930	10,365	8,924
24	629.2	16,500	2,380	14,120	11,889	2,770	9,119	7,851
25	628.2	14,300	2,270	12,030	10,129	2,550	7,579	6,526

1. U. S. Corps of Engineers temporary gage.
2. U. S. G. S. provisional records.

TABLE 4-25

SAINT JOHN RIVER AT LINCOLN SCHOOL, MAINE
COMPUTED MONTHLY AND ANNUAL DISCHARGE, IN C. F. S.
(Uncontrolled drainage area 1,647 square miles)

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1932	3,056	3,153	931	1,192	700	547	9,160	8,034	2,290	2,619	1,741	3,636	3,088
1933	2,658	3,523	1,627	759	526	387	4,647	13,762	3,396	1,941	574	488	2,857
1934	1,759	1,244	707	362	291	308	9,961	9,805	3,331	2,364	1,027	833	2,666
1935	1,386	2,696	2,514	916	543	477	5,108	7,274	4,750	2,301	719	559	2,437
1936	1,172	629	704	335	275	5,463	6,760	11,886	3,442	1,157	504	983	2,776
1937	2,122	2,186	1,153	1,600	1,023	974	5,413	9,964	3,406	1,125	1,990	1,253	2,684
1938	1,935	3,176	1,798	700	526	646	6,578	8,091	1,991	2,724	1,859	3,312	2,778
1939	2,267	1,429	2,503	903	365	357	1,771	12,914	4,439	3,121	4,238	1,572	2,990
1940	3,074	3,099	1,932	577	315	329	3,973	10,139	5,685	3,572	727	616	2,836
1941	415	2,213	909	1,296	584	468	7,234	4,494	2,555	2,013	1,458	1,625	2,105
1942	2,106	3,542	1,390	680	440	584	5,449	11,230	3,150	1,625	812	439	2,621
1943	786	699	380	257	189	255	1,153	12,849	4,529	2,475	1,511	911	2,166
1944	2,302	4,230	756	323	263	235	877	6,974	1,779	1,387	1,067	1,697	1,824
1945	2,857	1,279	729	801	557	1,133	12,125	6,398	3,167	3,631	1,315	1,557	2,962
1946	2,418	1,376	719	594	519	978	4,953	11,758	2,127	2,025	1,186	704	2,446
1947	1,430	1,763	1,195	601	1,120	1,062	4,197	16,638	6,054	2,330	2,231	747	3,281
1948	440	367	376	242	155	338	4,419	9,448	2,102	1,954	1,146	468	1,788
1949	1,159	2,176	2,042	877	634	974	7,615	5,230	2,685	1,043	461	582	2,126
1950	454	991	1,115	1,385	870	833	4,793	5,641	3,022	1,752	908	1,039	1,900
1951	871	3,528	6,048	1,412	1,365	1,000	9,623	6,297	3,033	2,448	1,106	839	3,131
1952	729	2,854	1,892	868	647	562	4,997	10,795	4,715	1,395	533	338	2,527
1953	1,125	1,117	859	683	840	1,353	11,293	7,624	1,708	876	310	377	2,347
1954	416	970	2,192	1,012	616	1,193	7,474	9,189	5,405	4,858	4,285	4,195	3,484

TABLE 4-26

SAINT JOHN RIVER AT LINCOLN SCHOOL
OBSERVED STAGES AND COMPUTED RATES OF FLOW

Date	Water surface elev.[1] ft. m. s. l.	Discharge in c. f. s. At Fort Kent below Fish River[2]	Fish R. at Fort Kent[2]	Net	Lincoln School
1957					
Oct. 21	538.1	3,410	207	3,203	2,697
22	538.3	4,930	204	4,726	3,979
23	538.0	4,810	200	4,610	3,882
24	537.8	4,340	207	4,133	3,480
25	538.1	4,850	275	4,575	3,852
26	538.4	7,600	267	7,333	6,174
27	539.2	9,160	255	8,905	7,498
28	538.6	7,800	255	7,545	6,353
29	538.4	6,580	251	6,329	5,329
30	538.2	5,640	255	5,385	4,534
31	538.0	5,050	259	4,791	4,034
Nov. 1	537.8	4,720	259	4,461	3,756
2	538.4	4,470	263	4,207	3,542
3	539.2	4,590	301	4,289	3,611
4	539.9	6,760	443	6,317	5,319

1. U. S. Corps of Engineers temporary gage.
2. U. S. G. S. provisional records.

TABLE 4-27

FRESH WATER SUPPLY FOR CONSTRUCTION AND OPERATION

Site	Location	Drainage area in acres
Letite gates and Letite lock	Small pond near Mascarene, New Brunswick, about 4 miles NE of site	935
Deer gates	Mill Creek, about 1/2 mile N. of site on Deer Island	1,014
Deer gates	Big Pond, about 4 miles N. of site on Deer Island	650
Campobello lock	Small stream about $1\frac{1}{2}$ miles SW of site on Campobello Island	160
Campobello lock	Small stream in marsh about $2\frac{1}{2}$ miles SW of site on Campobello Island	190
Quoddy lock	Small intermittent stream and marsh area about $1\frac{1}{2}$ miles N. of site on Campobello Island	250
Powerhouse and Dog lock	Boyden Lake through the Eastport Water Company	8,500
Digdeguash powerhouse and spillway	Lily Lake about $2\frac{1}{2}$ miles SW of site in New Brunswick. During construction period fresh water can be obtained from the Digdeguash River	550
Calais powerhouse	Flowed Land Ponds about $1\frac{1}{2}$ miles S. of site.	250
Lincoln School powerhouse and spillway	Adequate supply of fresh water available in Saint John River	-
Rankin Rapids powerhouse and spillway	Adequate supply of fresh water available in Saint John River	-
Big Rapids powerhouse and spillway	Adequate supply of fresh water available in Saint John River	-

The average fresh water yield per acre of drainage area is about 1250 g.p.d. (0.87 g.p.m.). This is based on a net annual runoff rate of 16.8 inches for a dry year and does not include storage losses of which the principal loss would be from seepage.

APPENDIX 4

PLATES

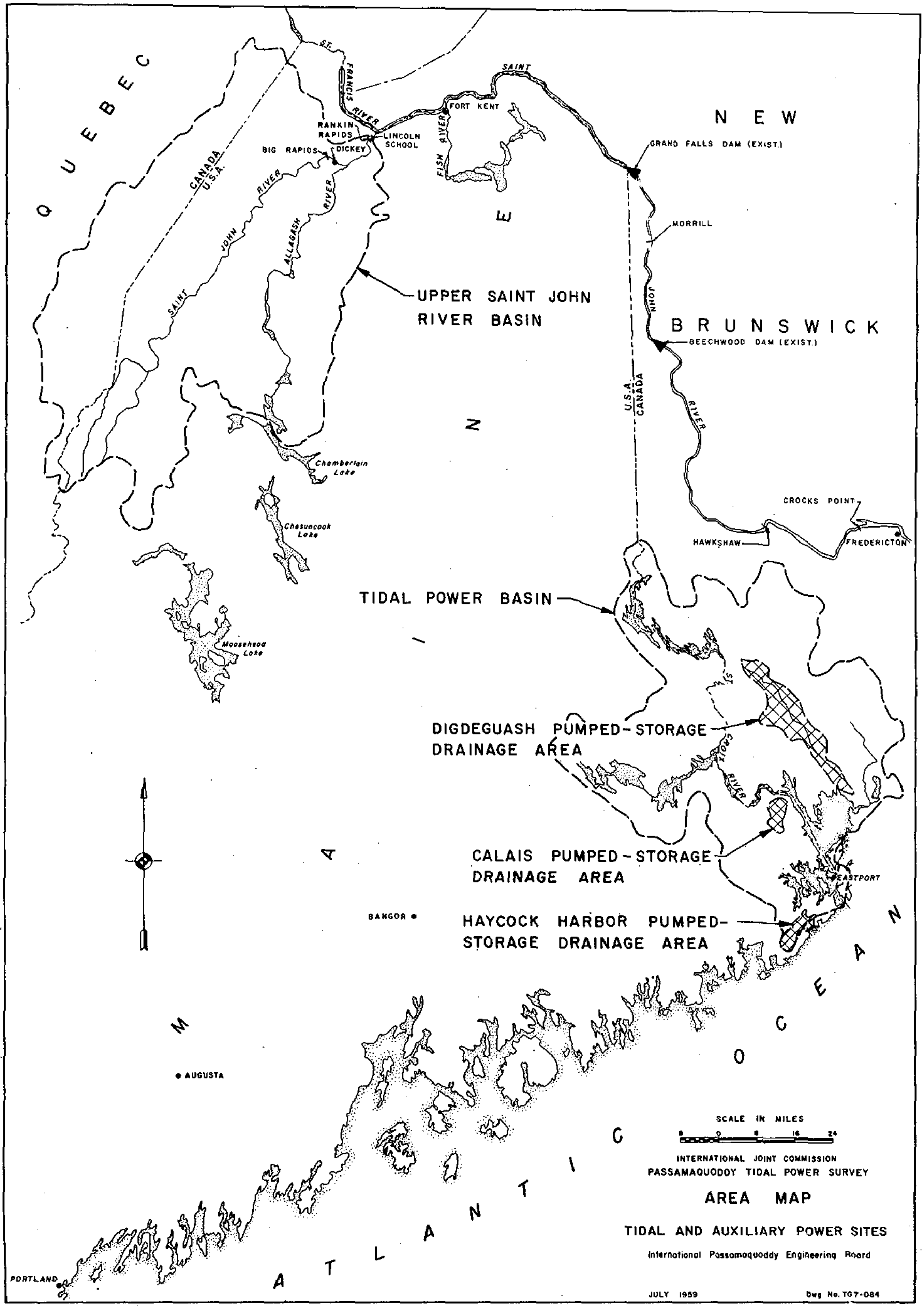
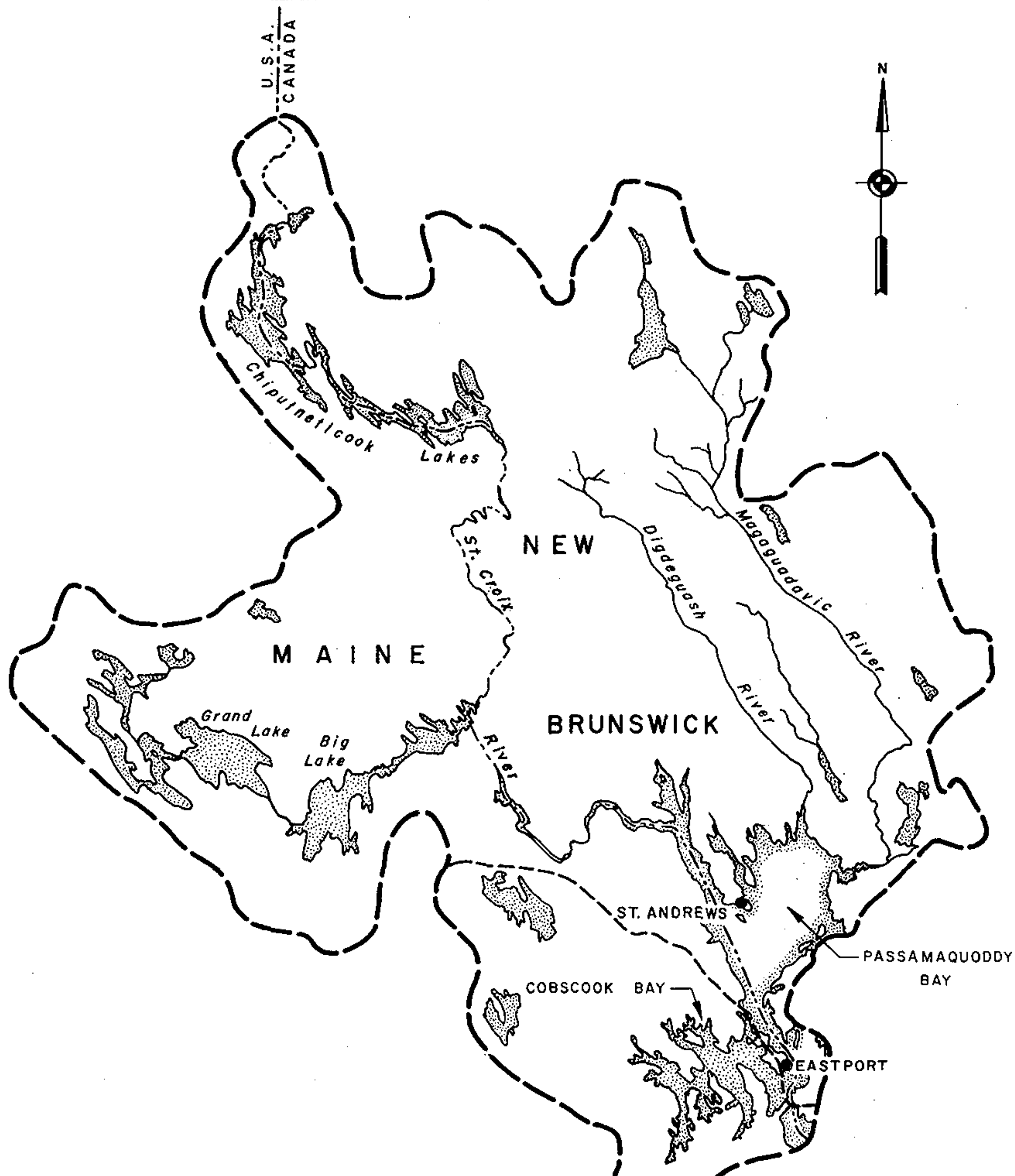
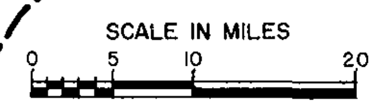

QUEBEC
CANADA
U.S.A.
ST. FRANCIS RIVER
SAINT
RANKIN RAPIDS
LINCOLN SCHOOL
BIG RAPIDS
DICKEY
FORT KENT
FISH RIVER
SAINT JOHN RIVER
ALLAGASH RIVER
NEW
BRUNSWICK
GRAND FALLS DAM (EXIST.)
MORRILL
JOHN
BEECHWOOD DAM (EXIST.)
RIVER
U.S.A.
CANADA
UPPER SAINT JOHN
RIVER BASIN
Chamberlain Lake
Chesuncook Lake
Moosehead Lake
CROCKS POINT
HAWKSHAW
FREDERICTON
TIDAL POWER BASIN
ST. CROIX RIVER
DIGDEGUASH PUMPED-STORAGE
DRAINAGE AREA
CALAIS PUMPED-STORAGE
DRAINAGE AREA
EASTPORT
HAYCOCK HARBOR PUMPED-
STORAGE DRAINAGE AREA
BANGOR
AUGUSTA
PORTLAND
M A I N E
ATLANTIC OCEAN
SCALE IN MILES
8 0 8 16 24
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
AREA MAP
TIDAL AND AUXILIARY POWER SITES
International Passamaquoddy Engineering Board
JULY 1959
Dwg No. TG7-084

PLATE 4-1





DRAINAGE AREAS

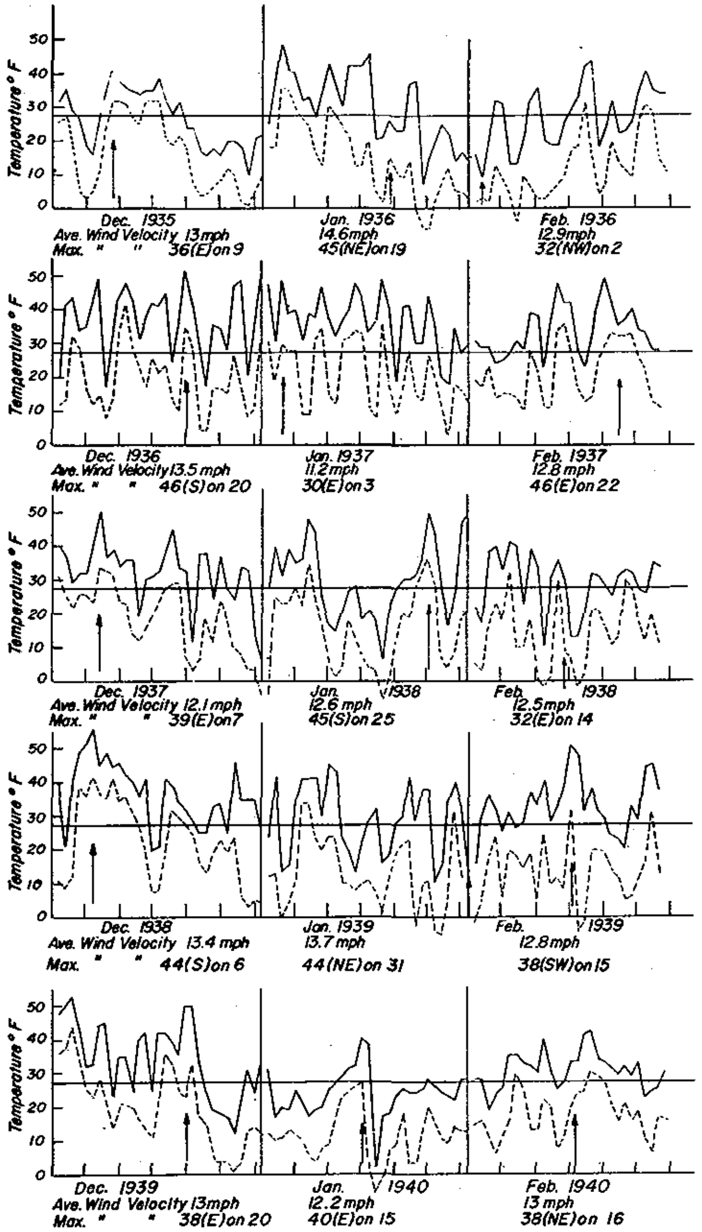
LOCATION	AREA IN SQUARE MI.
PASSAMAQUODDY BAY (UPPER POOL)	2,646
COBSCOOK BAY (LOWER POOL)	407
TOTAL (TIDAL PROJECT)	3,053

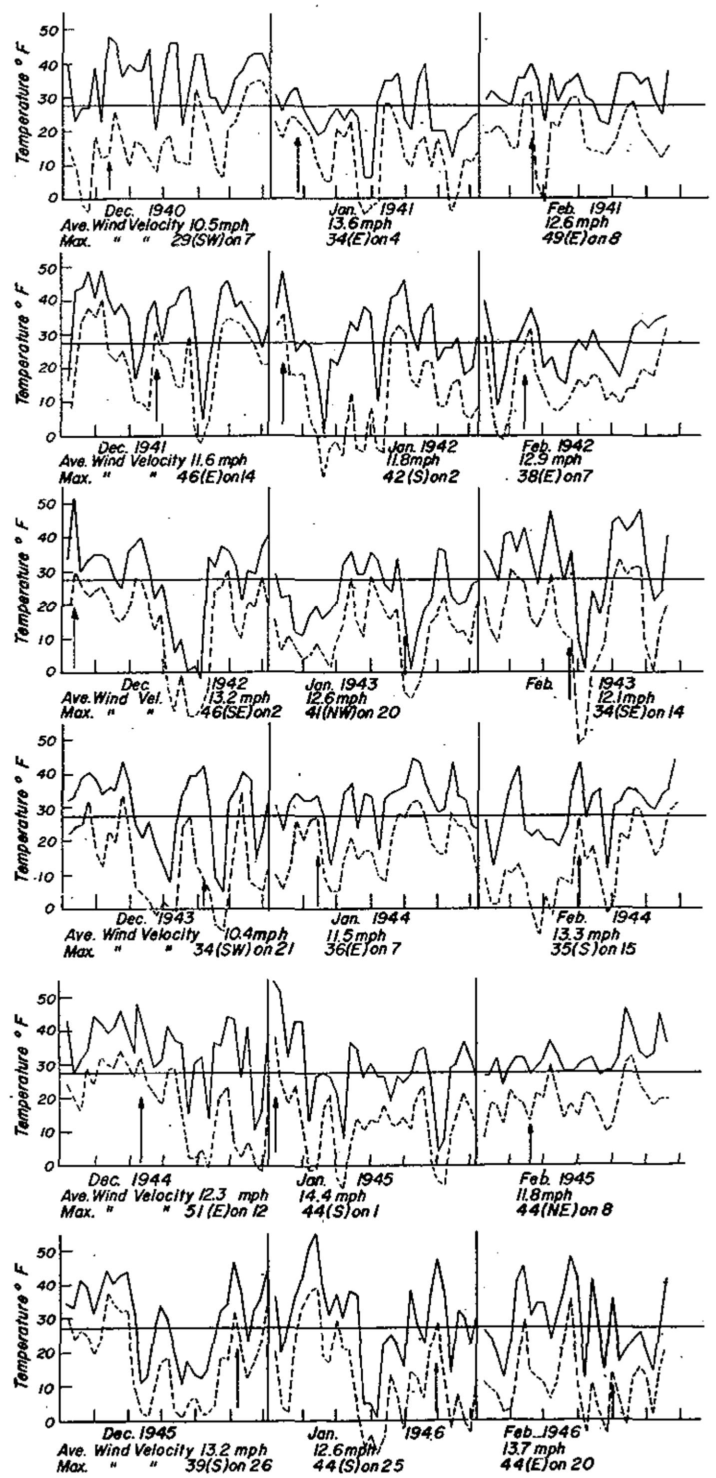
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

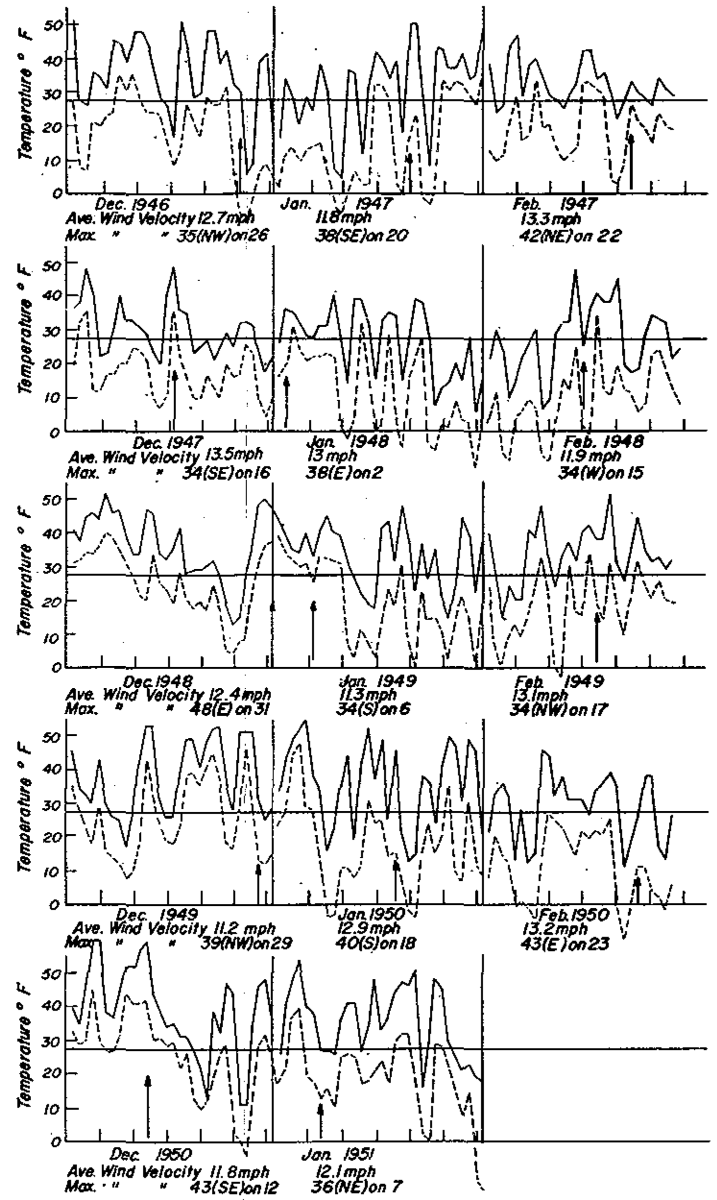
TIDAL POWER PROJECT
DRAINAGE AREA

International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG 7-085







NOTES:

The freezing point of sea water is 27.5°F

↑ *Indicates date of maximum wind velocity*

——— *Daily maximum temperatures*

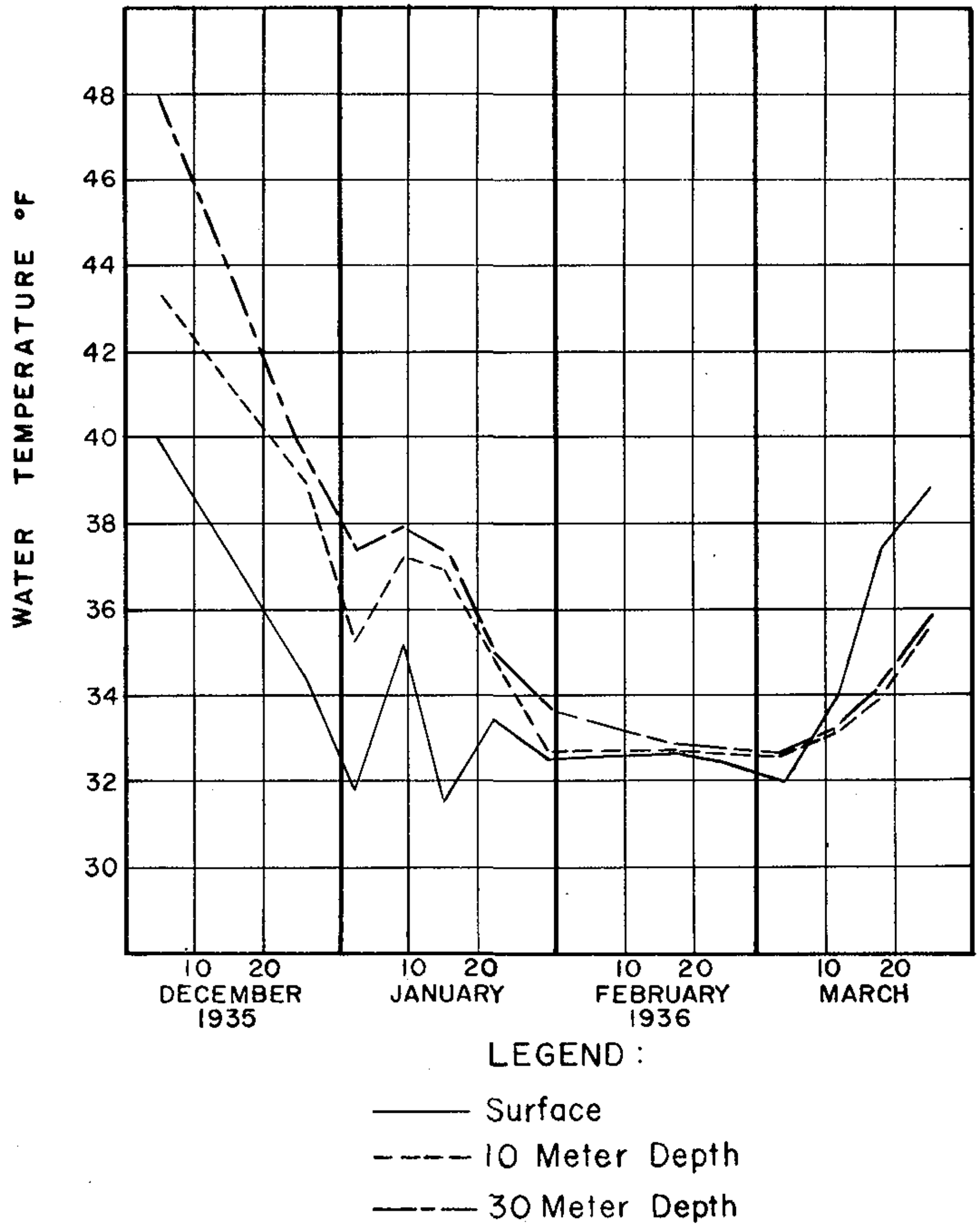
- - - - *Daily minimum temperatures*

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

TIDAL POWER PROJECT
AIR TEMPERATURES
DEC. - FEB., 1935 - 51
EASTPORT, MAINE

International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG 7-086



INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

TIDAL POWER PROJECT

WATER TEMPERATURES

ST. ANDREWS, NEW BRUNSWICK

International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG 7-087

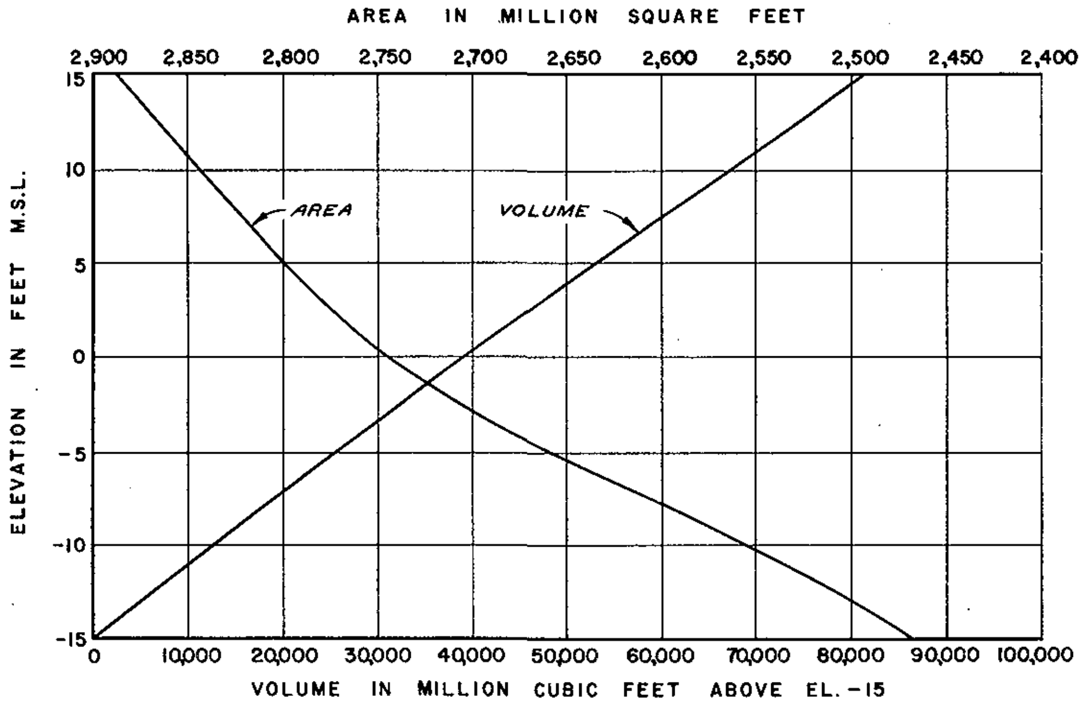


FIG. 1 - HIGH POOL

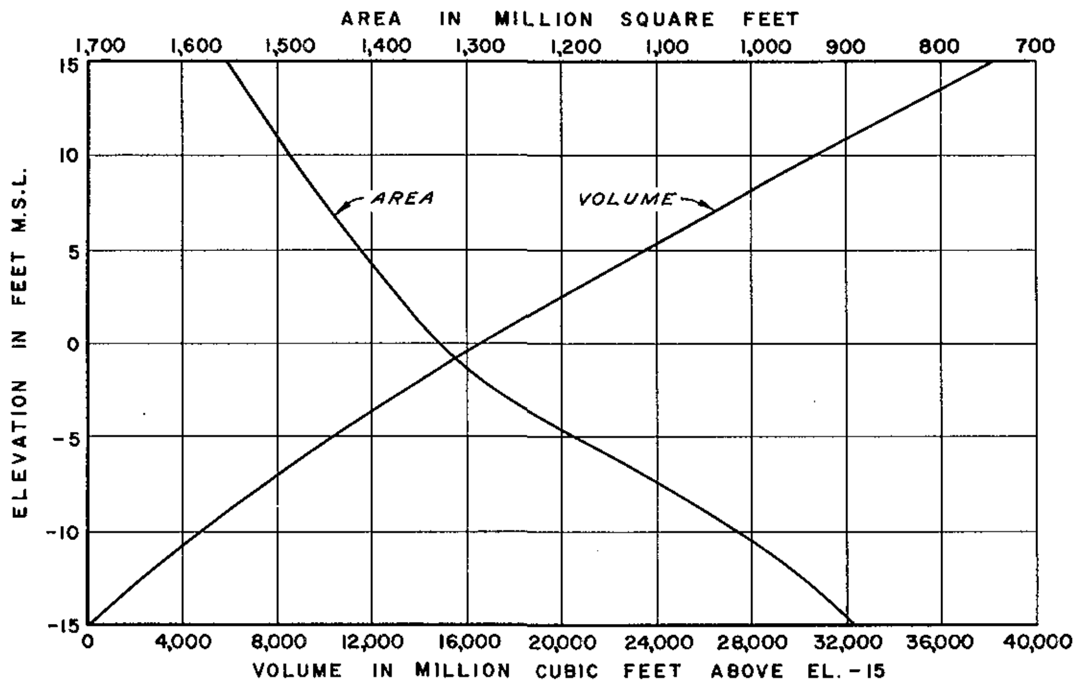


FIG. 2 - LOW POOL

NOTE:
AREA & CAPACITY CURVES ARE FOR STUDY 4-6.22 AND 4-6.33

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

TIDAL POWER PROJECT

AREA & CAPACITY CURVES
SELECTED PLAN

International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG7-088

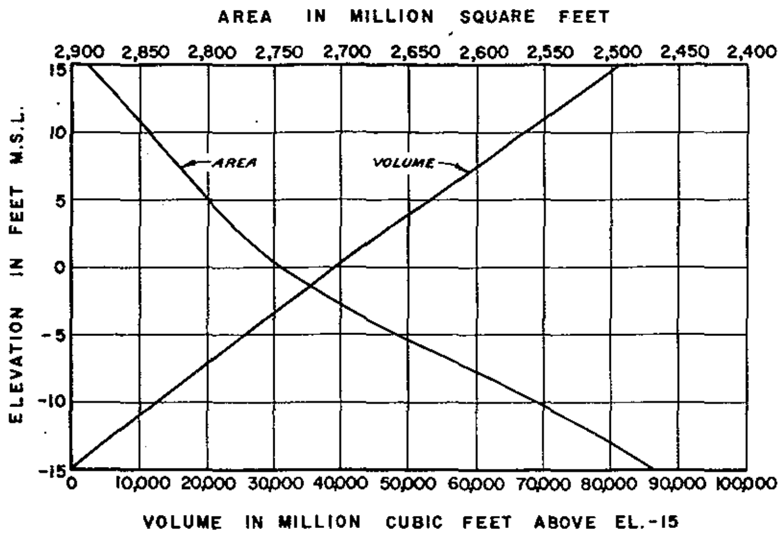


FIG. 1 – HIGH POOL
(STUDY 7B-7.212)

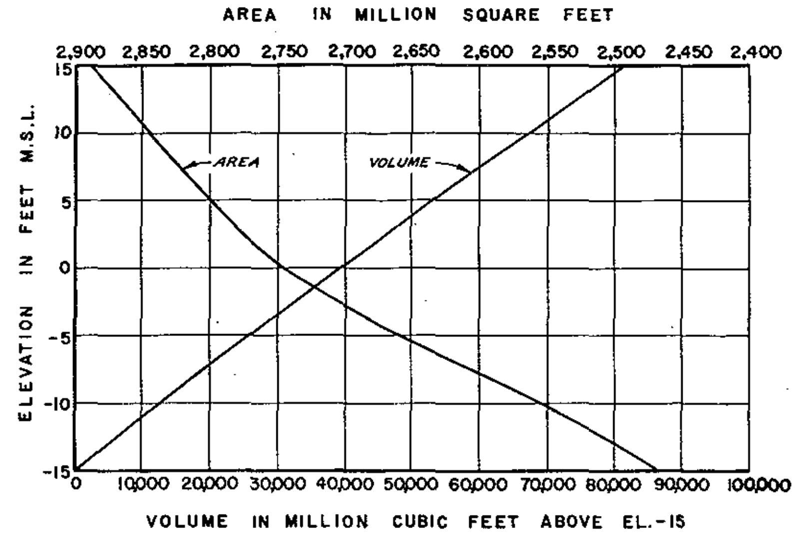


FIG. 3 – HIGH POOL
(STUDY 7B-7.22)

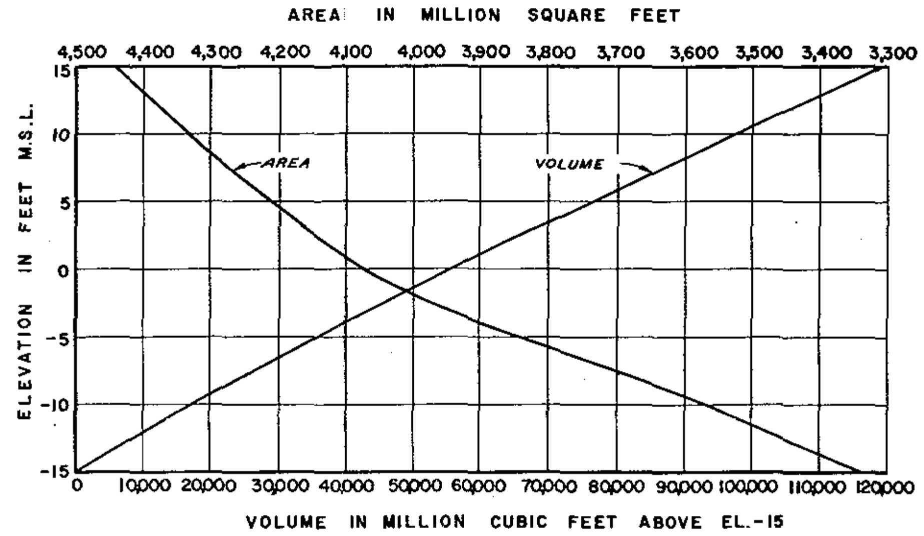


FIG. 5 – SINGLE POOL
(HIGH AND LOW POOLS OF STUDY 4-6.33 COMBINED)

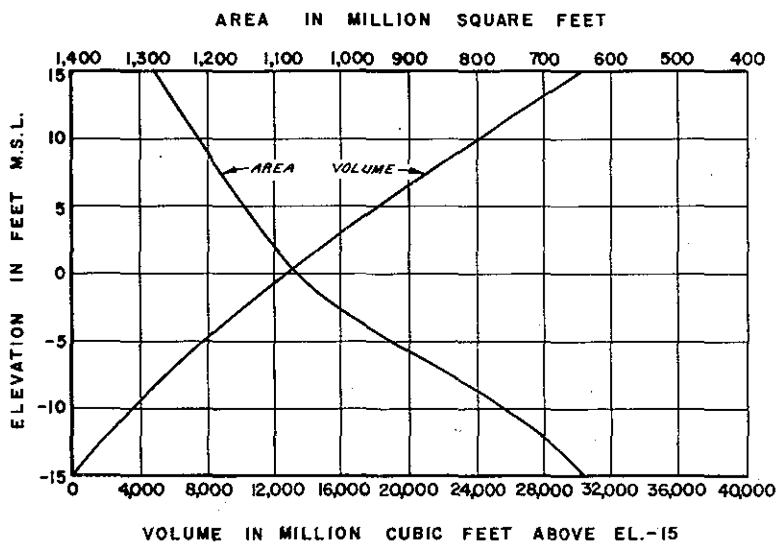


FIG. 2 – LOW POOL
(STUDY 7B-7.212)

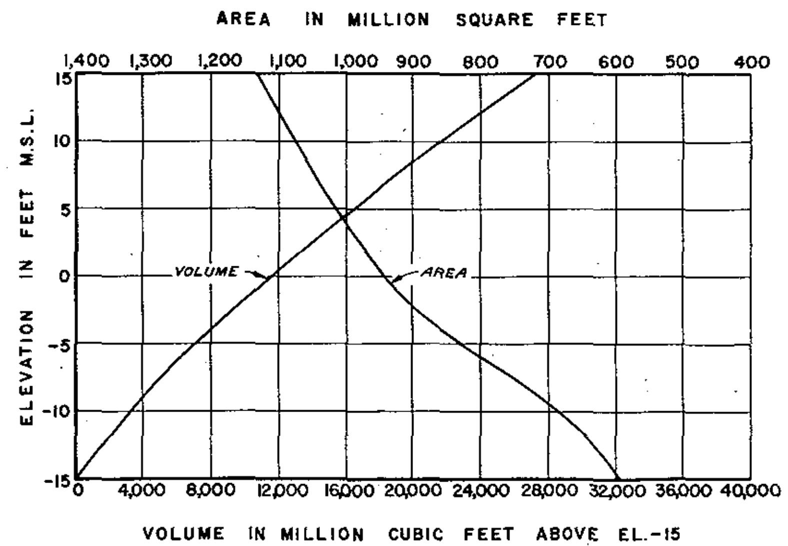


FIG. 4 – LOW POOL
(STUDY 7B-7.22)

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

TIDAL POWER PROJECT
AREA&CAPACITY CURVES

OTHER PLANS CONSIDERED
SHEET 1
International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG7-089

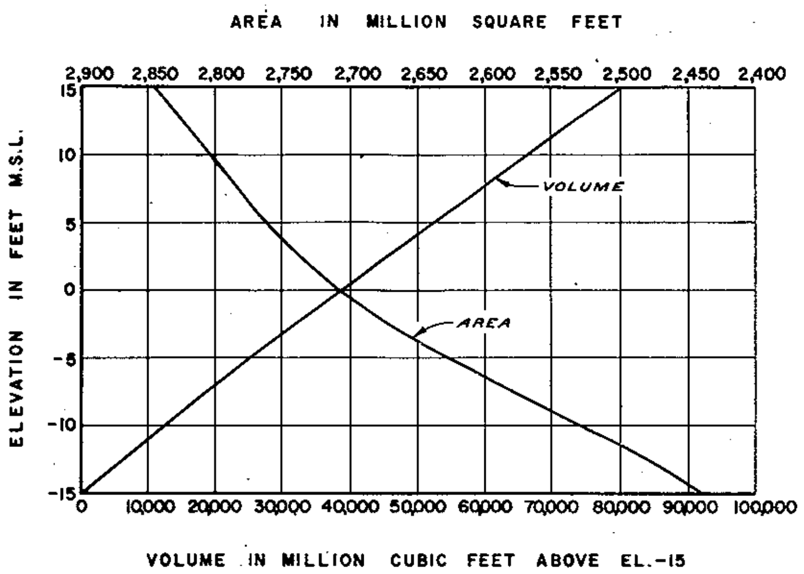


FIG. 1 - HIGH POOL
(STUDY 1-1.211)

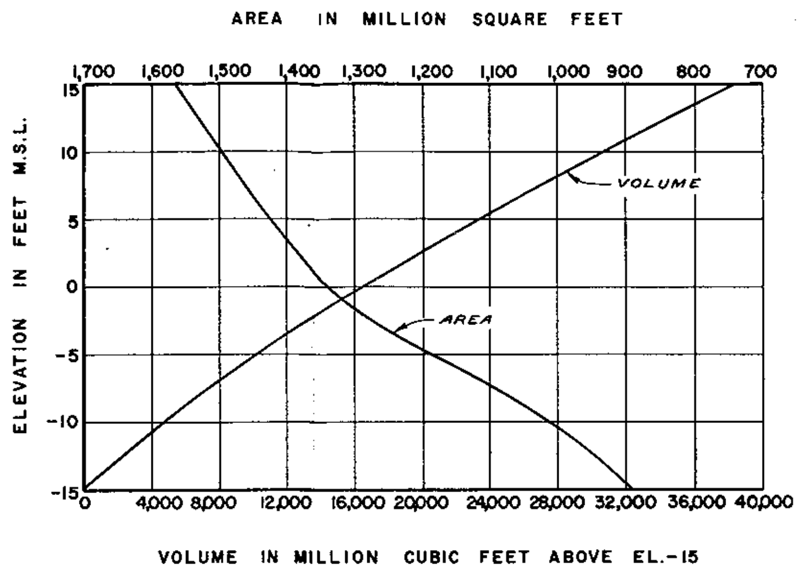


FIG. 3 - HIGH POOL
(STUDY 4-6.52)

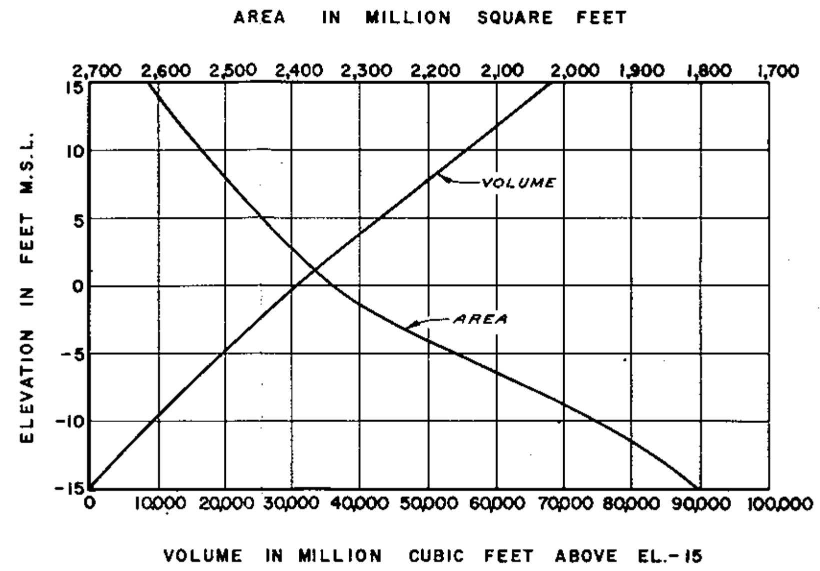


FIG. 5 - HIGH POOL
(STUDY 6A-2.613)

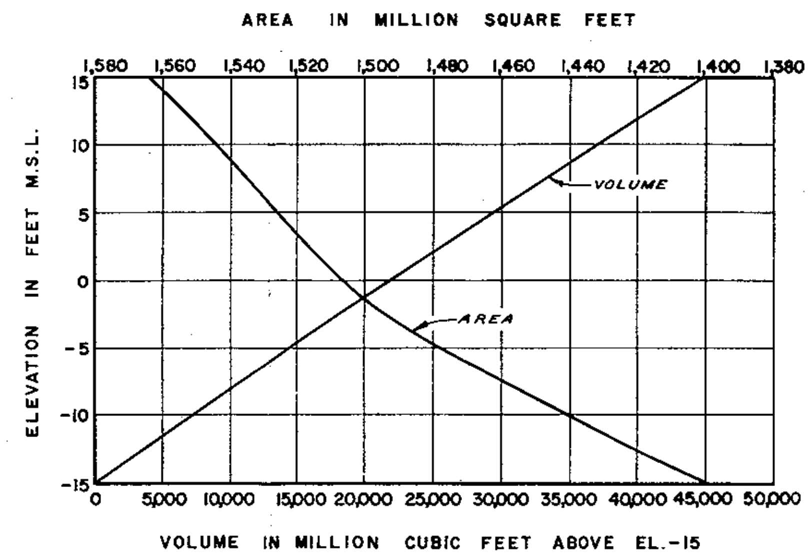


FIG. 6 - LOW POOL
(STUDY 6A-2.613)

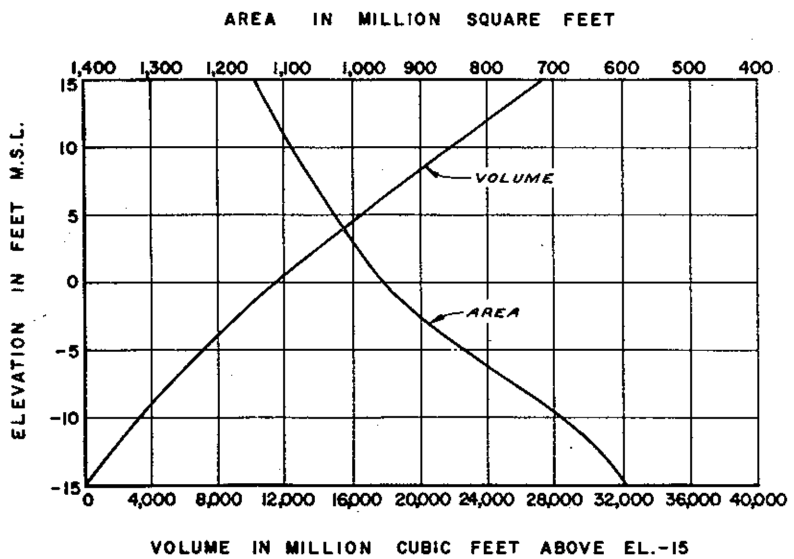


FIG. 2 - LOW POOL
(STUDY 1-1.211)

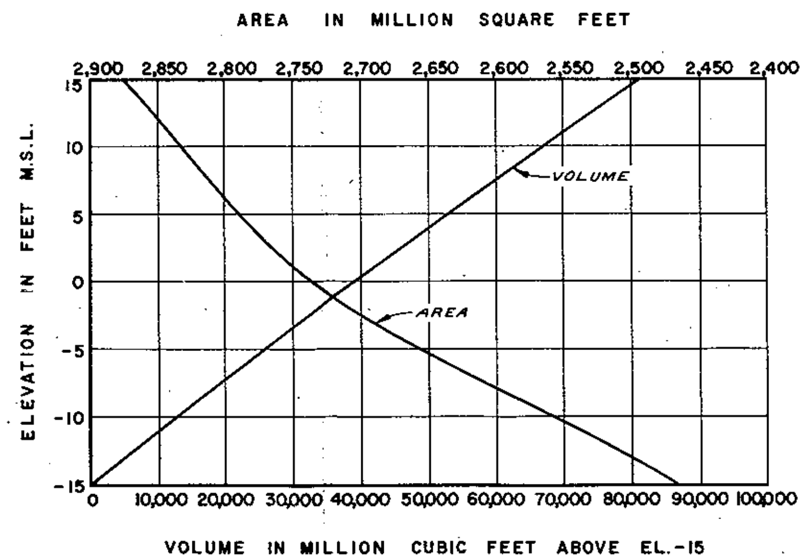
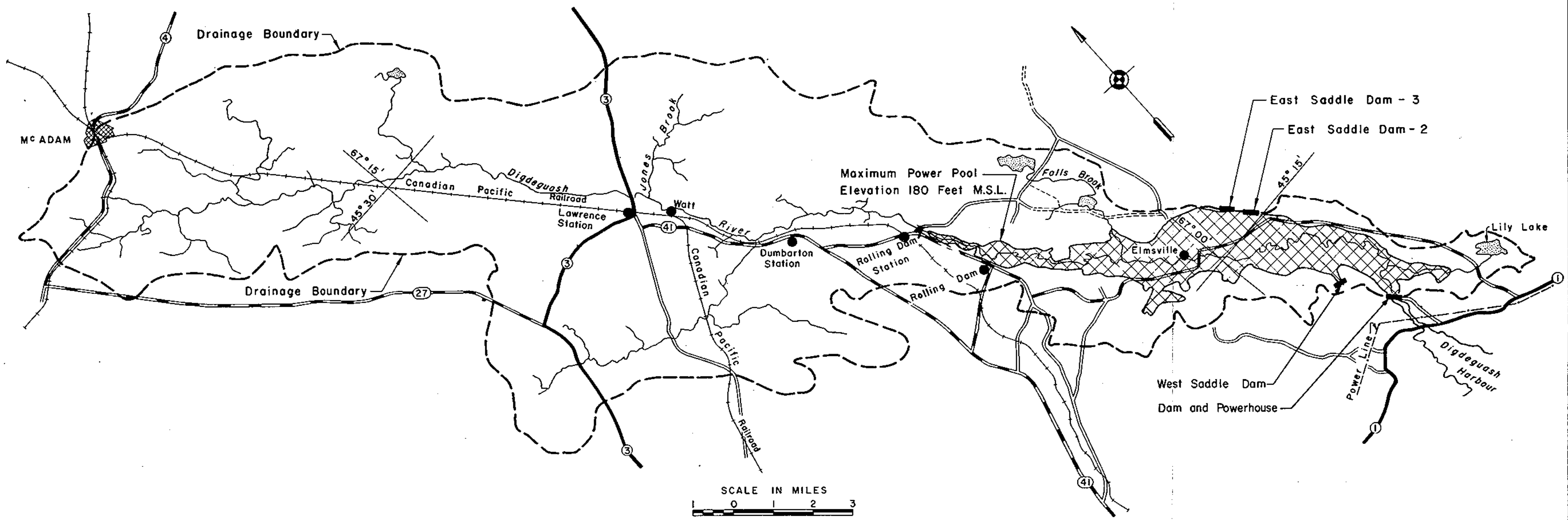


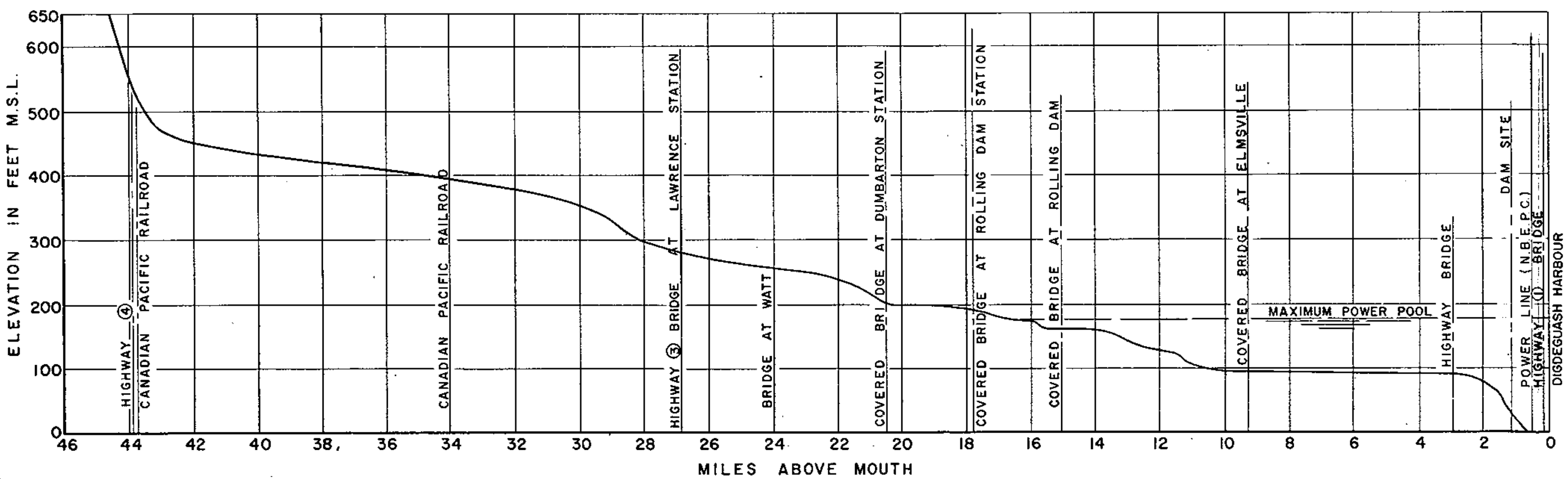
FIG. 4 - LOW POOL
(STUDY 4-6.52)

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
TIDAL POWER PROJECT
AREA&CAPACITY CURVES
OTHER PLANS CONSIDERED
SHEET 2
International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG7-090



RESERVOIR AREA



PROFILE

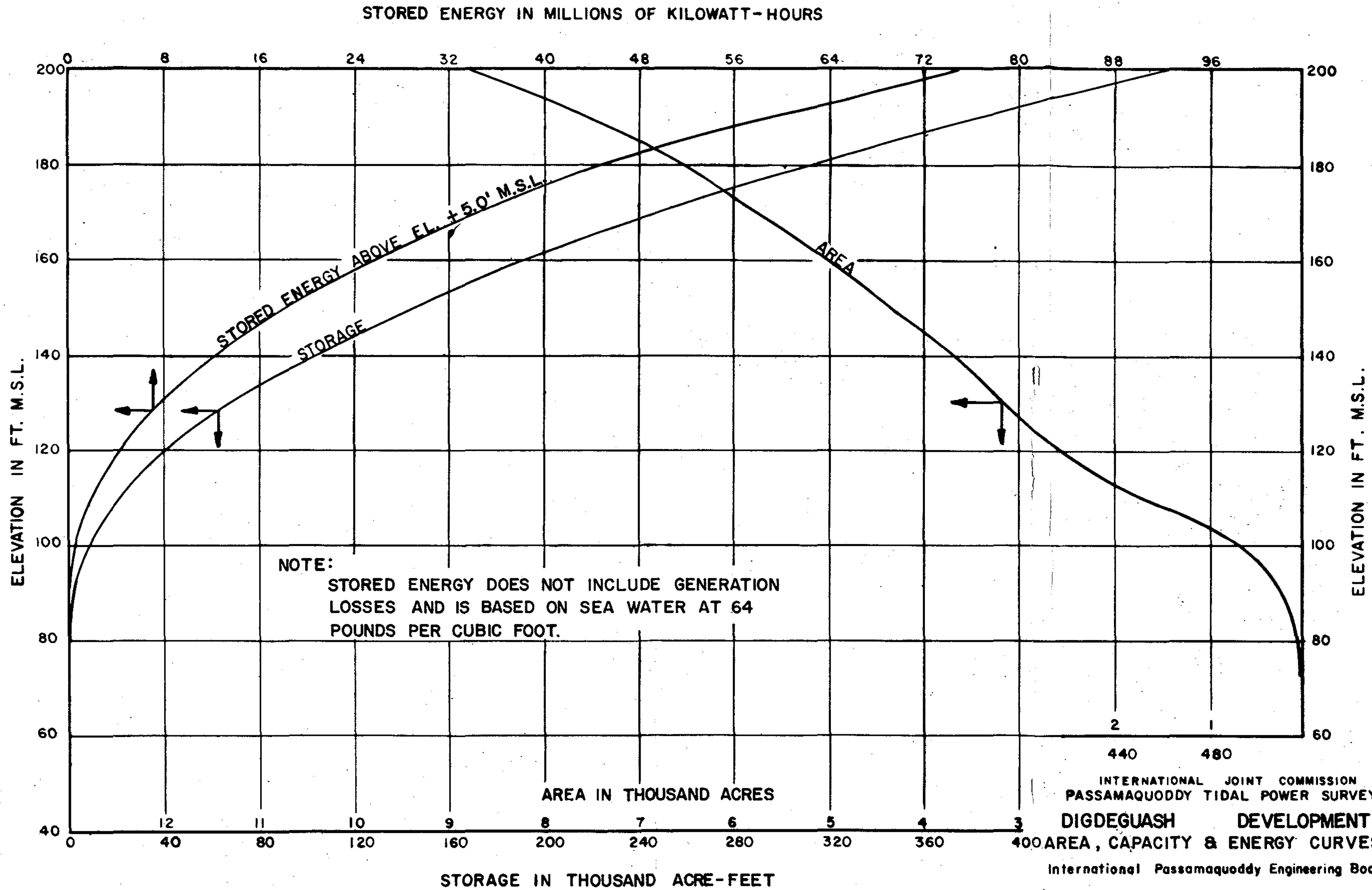
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

DIGEGUASH DEVELOPMENT

DRAINAGE AREA & RIVER PROFILE

International Passamaquoddy Engineering Board

JULY 1959 Dwg. No.TG7-091



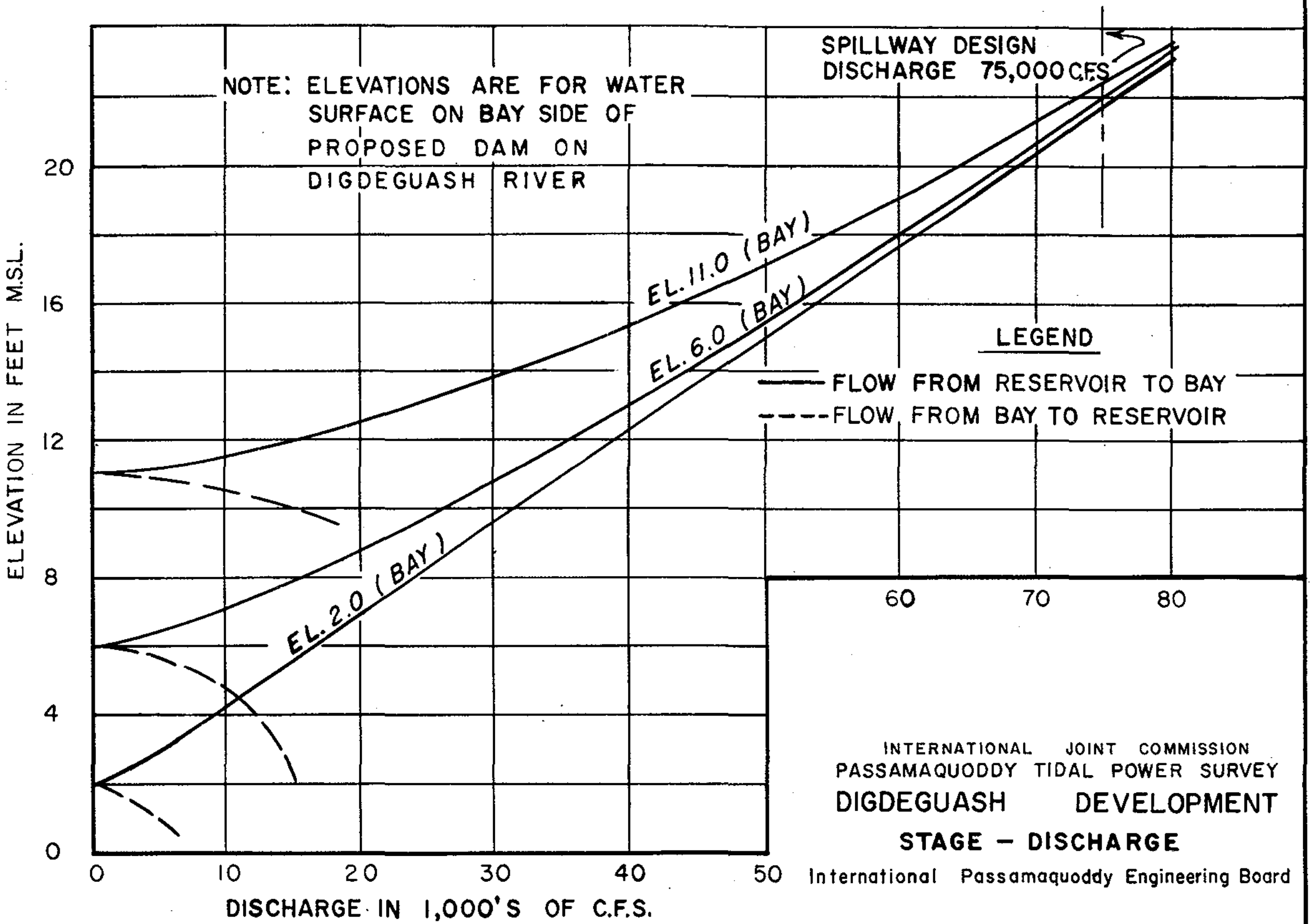
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

DIGDEGUASH DEVELOPMENT

AREA, CAPACITY & ENERGY CURVES

International Passamaquoddy Engineering Board

JULY 1959 Dwg. No TG7-092



INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

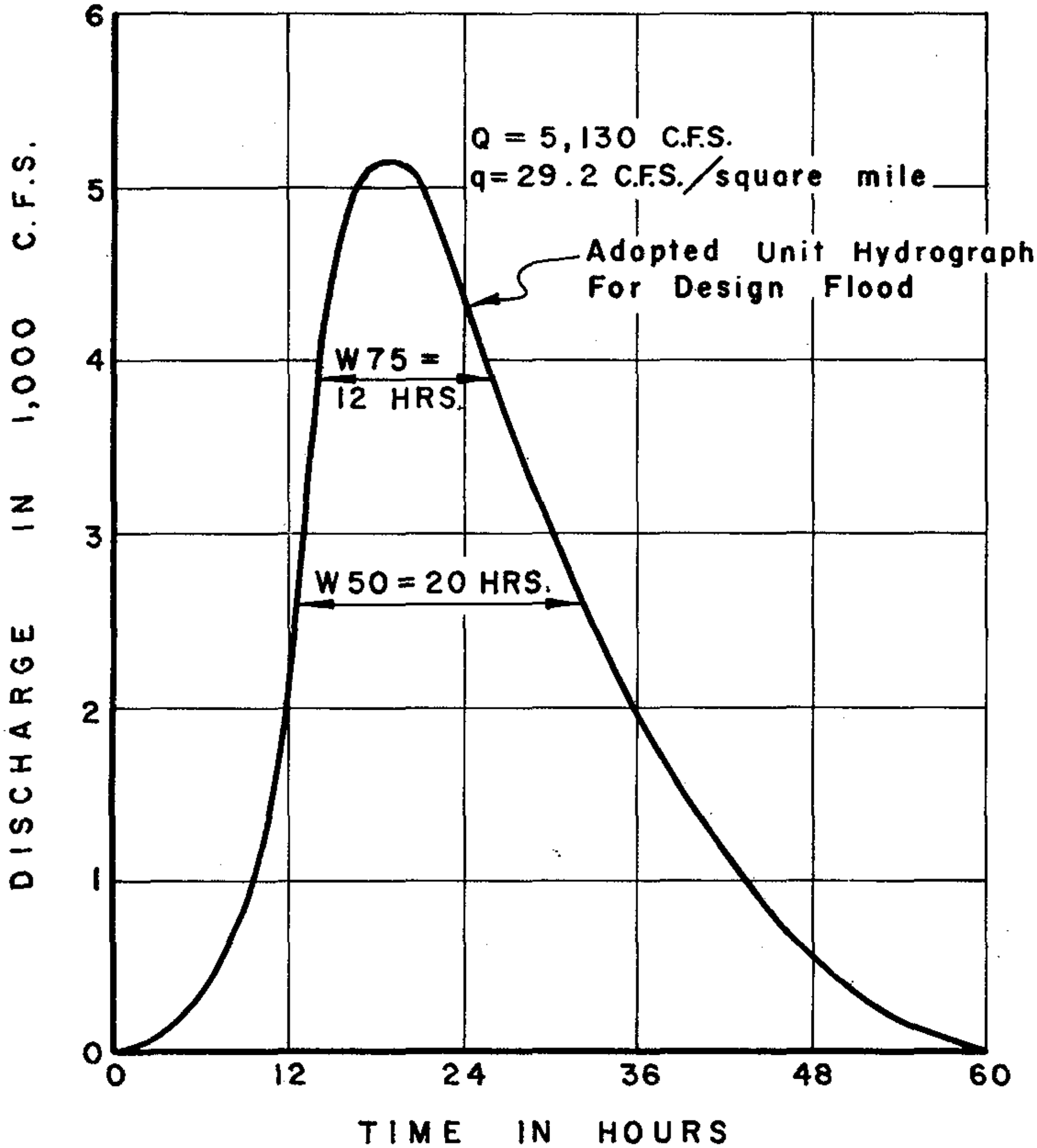
DIGDEGUASH DEVELOPMENT

STAGE – DISCHARGE

International Passamaquoddy Engineering Board

JULY 1959

Dwg. No. TG7-093



Note:

Hydrograph is for a 6-hour unit rainfall duration.

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
DIGDEGUASH DEVELOPMENT
UNIT HYDROGRAPH
International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG 7-094

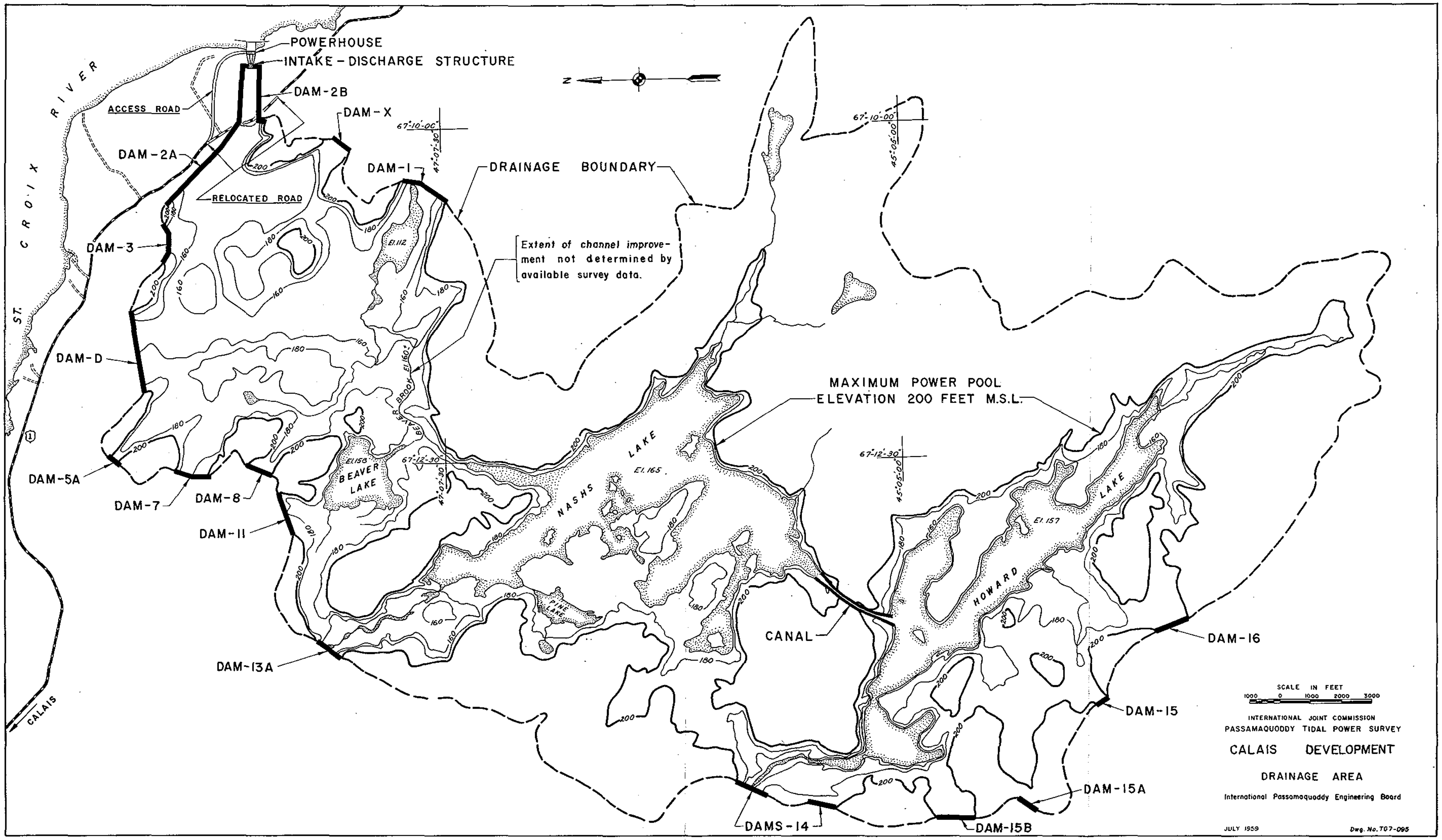
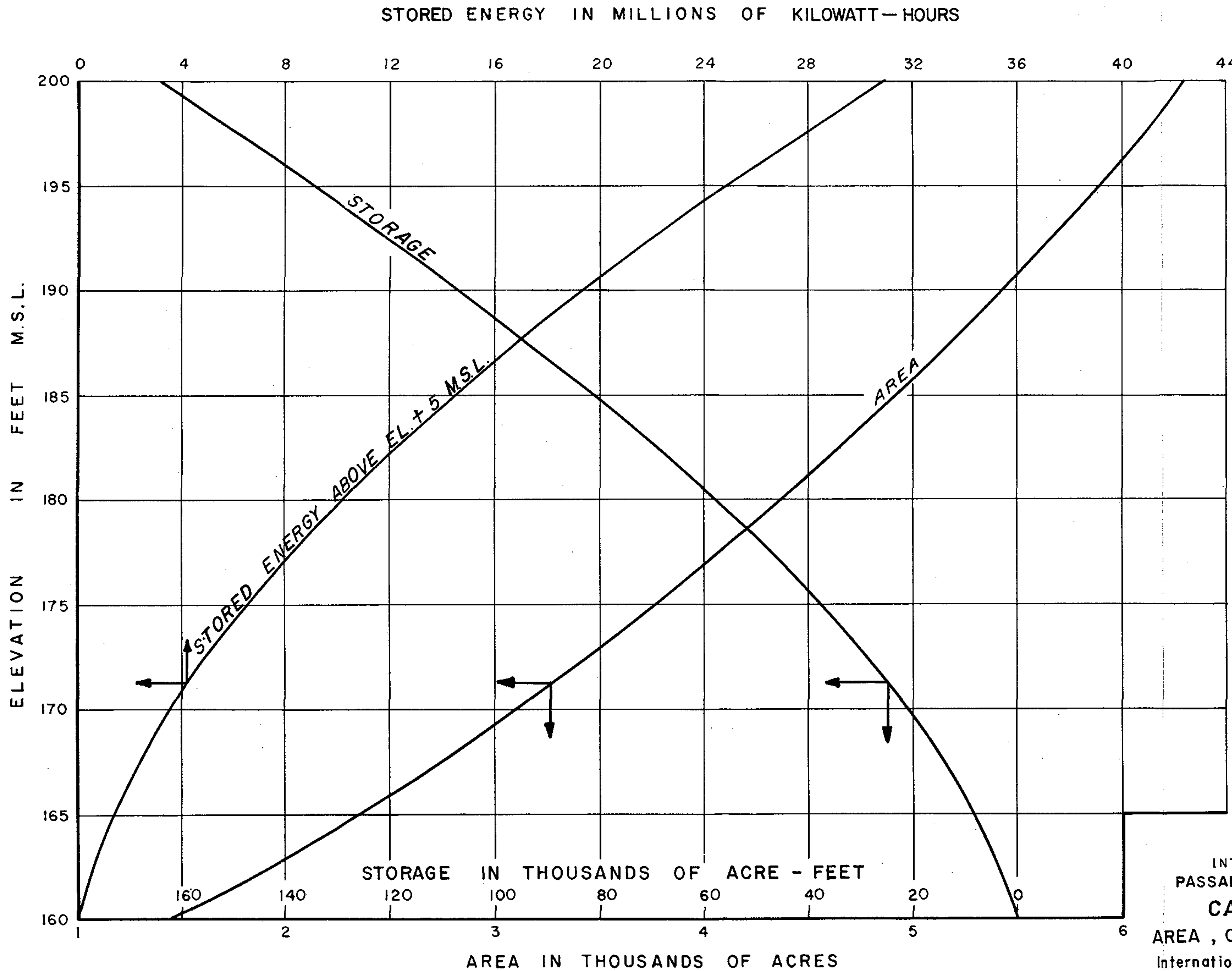

POWERHOUSE
INTAKE - DISCHARGE STRUCTURE
DAM-2B
DAM-X
ACCESS ROAD
DAM-2A
RELOCATED ROAD
DAM-1
DRAINAGE BOUNDARY
ST. CROIX RIVER
DAM-3
Extent of channel improvement not determined by available survey data.
DAM-D
MAXIMUM POWER POOL ELEVATION 200 FEET M.S.L.
DAM-5A
DAM-7
DAM-8
BEAVER LAKE
NASHS LAKE
HOWARD LAKE
PINE LAKE
DAM-11
CANAL
DAM-16
DAM-13A
DAM-15
CALAIS
DAM-15A
DAMS-14
DAM-15B
SCALE IN FEET
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
CALAIS DEVELOPMENT
DRAINAGE AREA
International Passamaquoddy Engineering Board
JULY 1959
Dwg. No. TG7-095

PLATE 4-12



NOTE:
STORED ENERGY DOES NOT INCLUDE GENERATION LOSSES AND IS BASED ON SEA WATER AT 64 POUNDS PER CUBIC FOOT.

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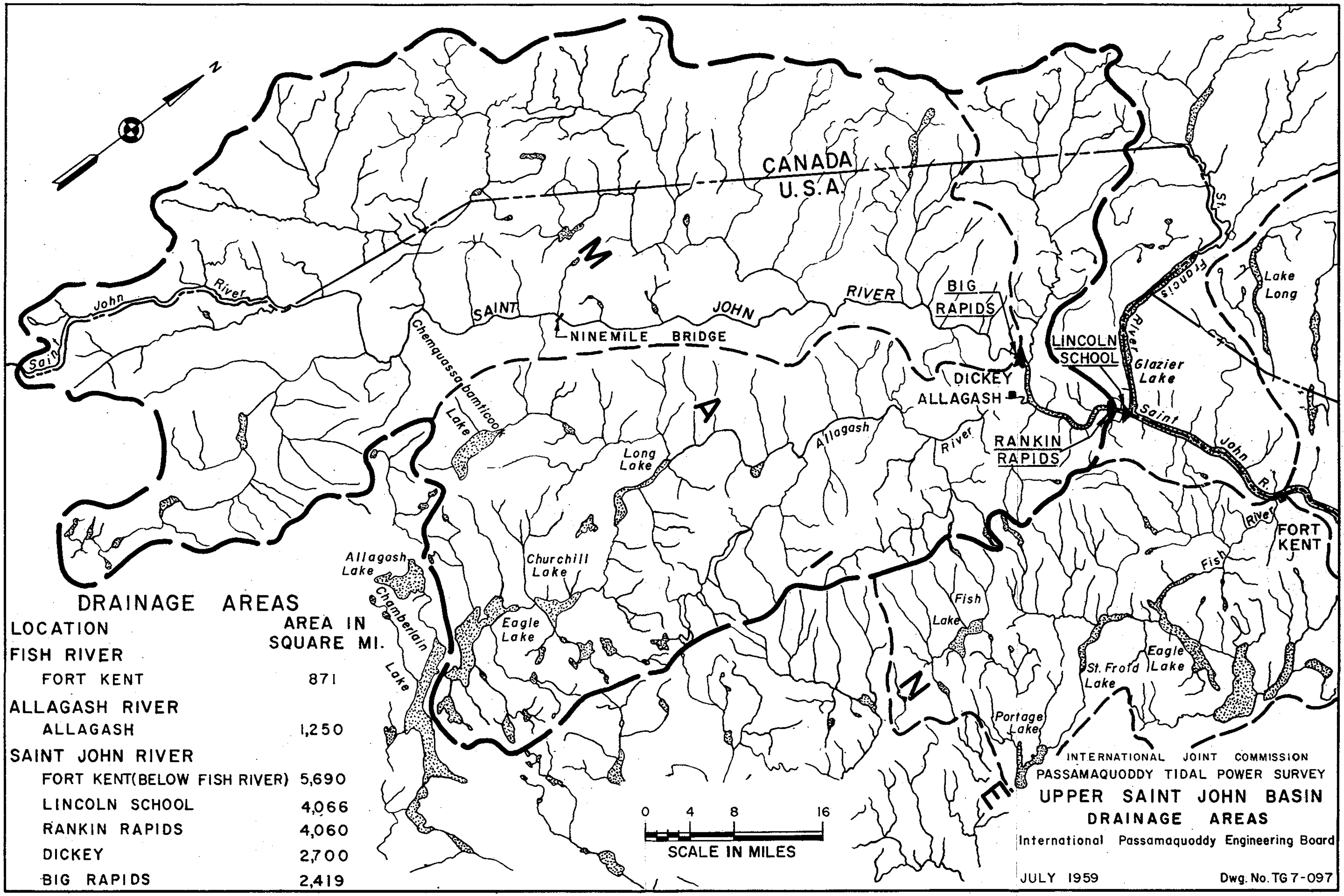
CALAIS DEVELOPMENT
AREA, CAPACITY & ENERGY CURVES

International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG 7-096



DRAINAGE AREAS

LOCATION	AREA IN SQUARE MI.
FISH RIVER	
FORT KENT	871
ALLAGASH RIVER	
ALLAGASH	1,250
SAINT JOHN RIVER	
FORT KENT (BELOW FISH RIVER)	5,690
LINCOLN SCHOOL	4,066
RANKIN RAPIDS	4,060
DICKEY	2,700
BIG RAPIDS	2,419

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

UPPER SAINT JOHN BASIN

DRAINAGE AREAS

International Passamaquoddy Engineering Board

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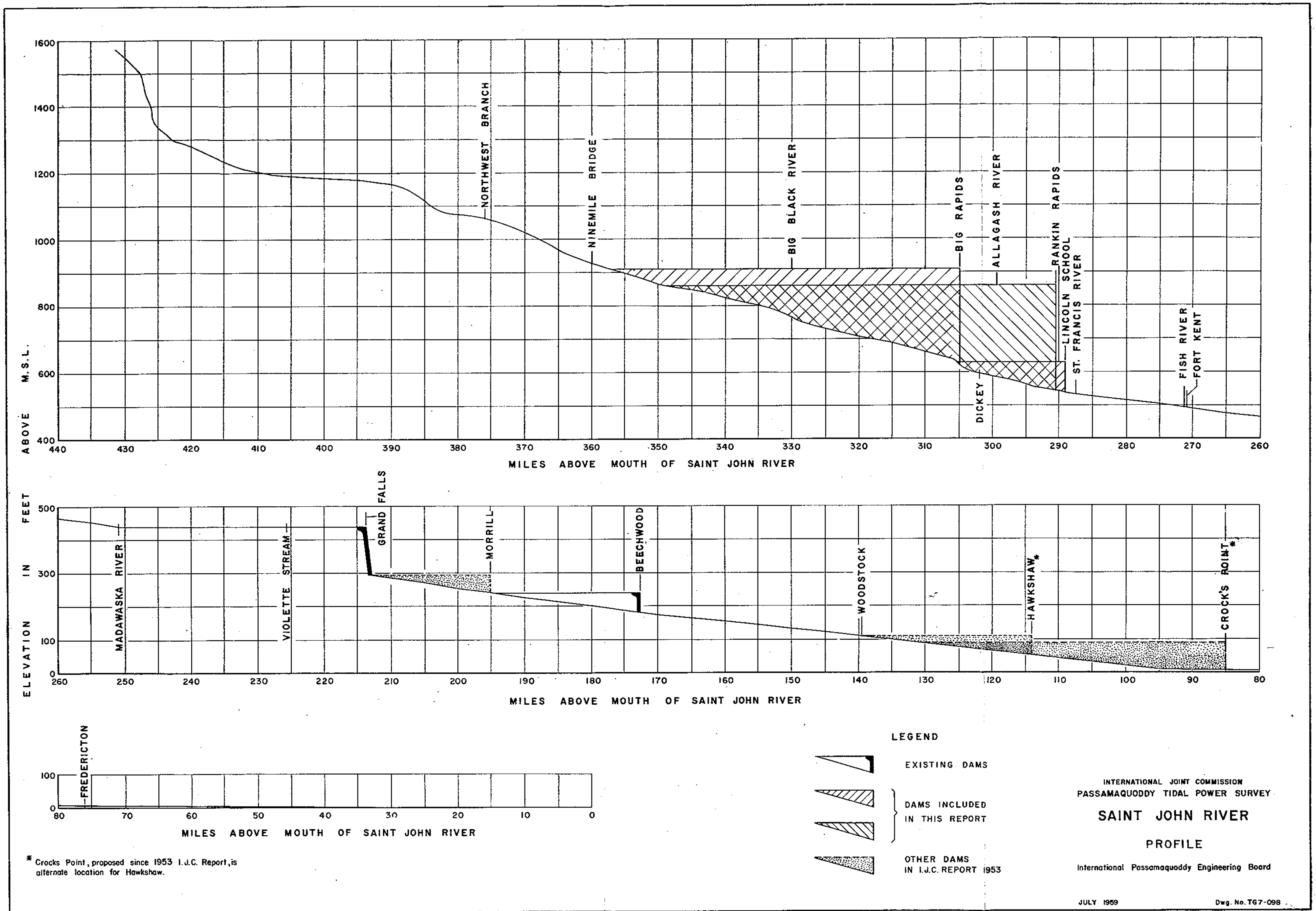


ELEVATION IN FEET ABOVE M.S.L.
MILES ABOVE MOUTH OF SAINT JOHN RIVER
NORTHWEST BRANCH
NINEMILE BRIDGE
BIG BLACK RIVER
BIG RAPIDS
ALLAGASH RIVER
RANKIN RAPIDS
LINCOLN SCHOOL
ST. FRANCIS RIVER
DICKEY
FISH RIVER
FORT KENT
MADAWASKA RIVER
VIOLETTE STREAM
GRAND FALLS
MORRILL
BEECHWOOD
WOODSTOCK
HAWKSHAW*
CROCK'S POINT*
FREDERICTON
LEGEND
EXISTING DAMS
DAMS INCLUDED IN THIS REPORT
OTHER DAMS IN I.J.C. REPORT 1953
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
SAINT JOHN RIVER
PROFILE
International Passamaquoddy Engineering Board
JULY 1959
Dwg. No. TG7-098
* Crocks Point, proposed since 1953 I.J.C. Report, is alternate location for Hawkshaw.

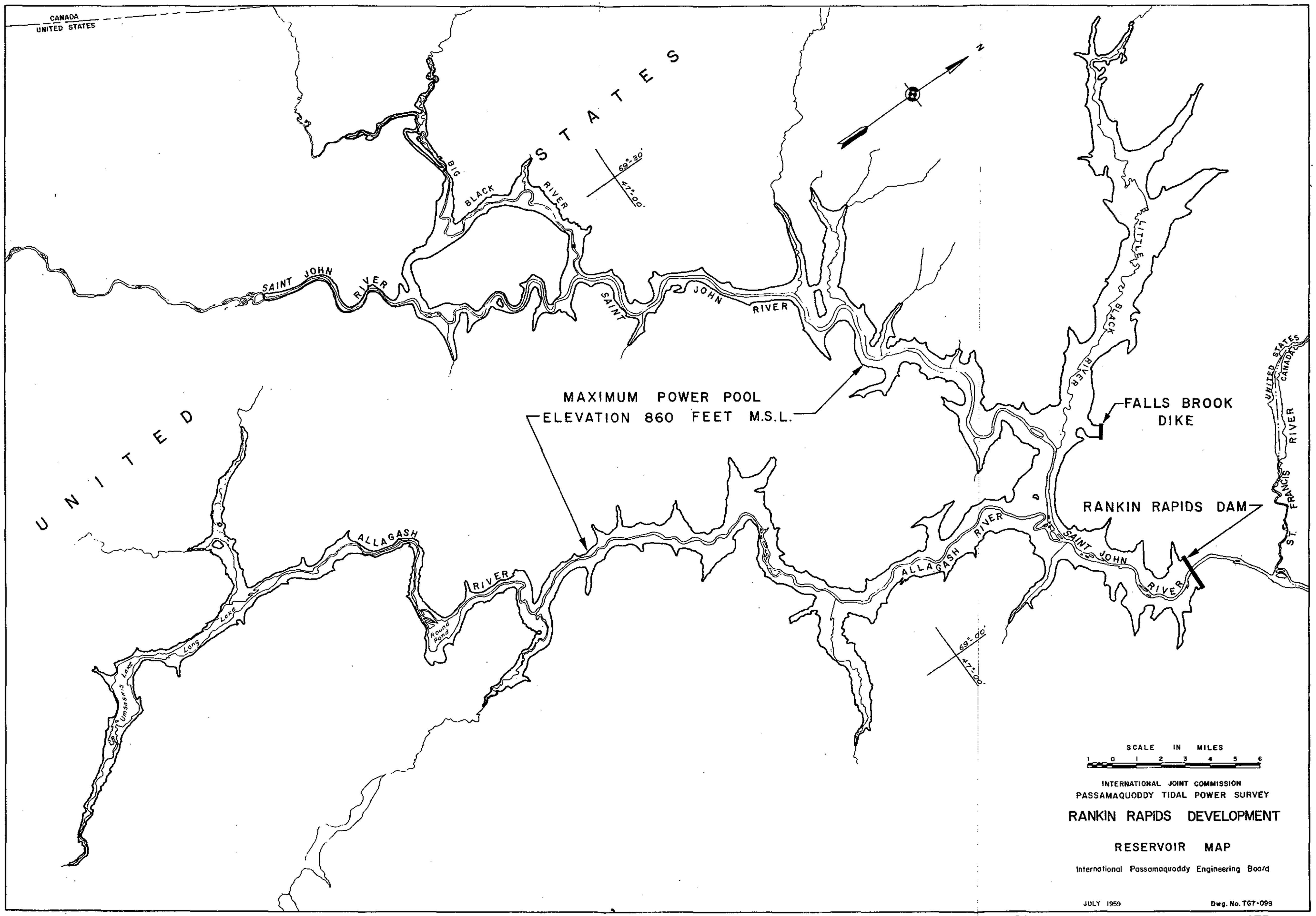
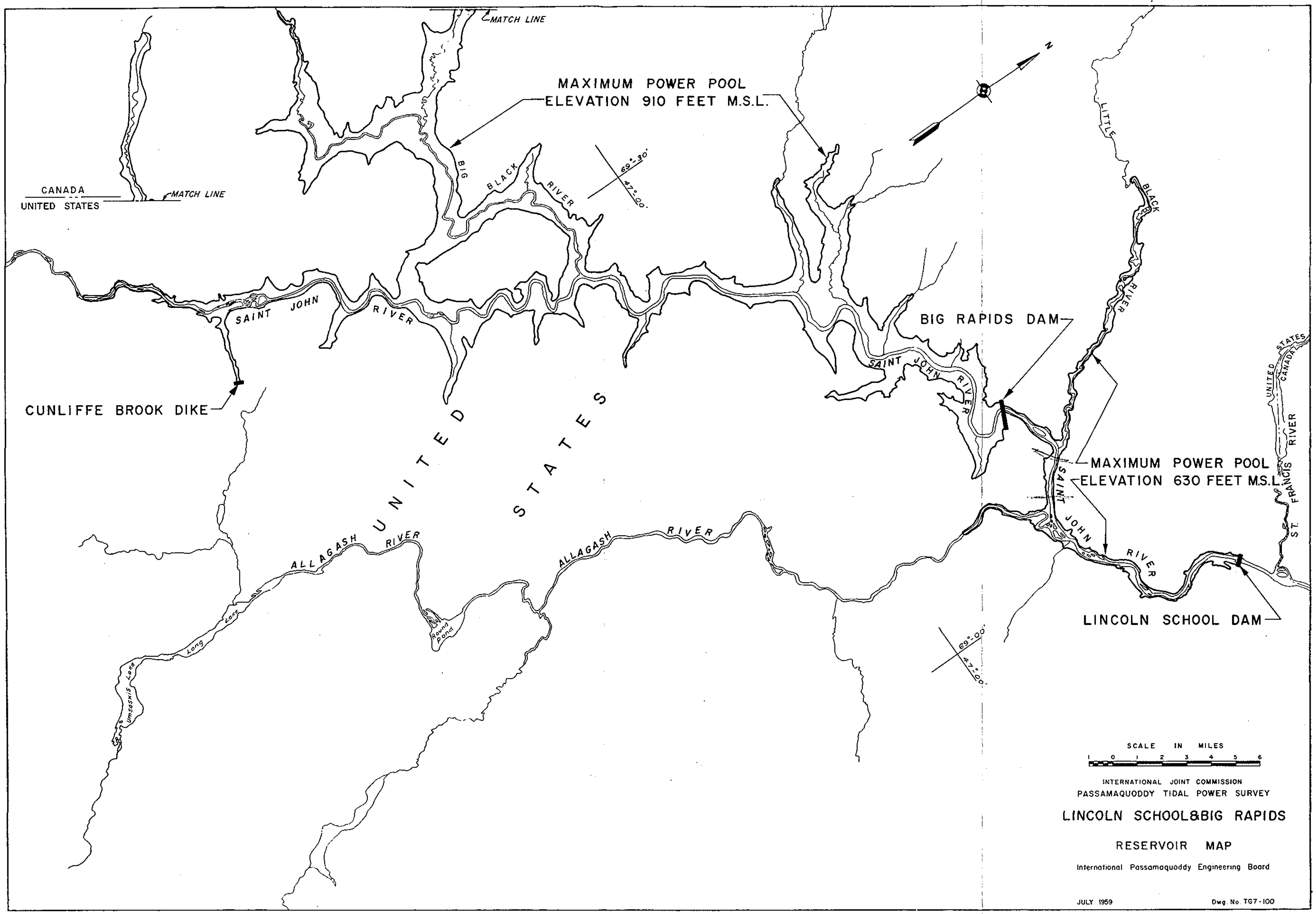

CANADA
UNITED STATES
UNITED STATES
BIG BLACK RIVER
SAINT JOHN RIVER
SAINT JOHN RIVER
69°30'
47°00'
N
LITTLE BLACK RIVER
BLACK RIVER
MAXIMUM POWER POOL
ELEVATION 860 FEET M.S.L.
FALLS BROOK DIKE
UNITED STATES
CANADA
ST. FRANCIS RIVER
RANKIN RAPIDS DAM
ALLAGASH RIVER
ALLAGASH RIVER
SAINT JOHN RIVER
Round Pond
Long Lake
Umsaskis Lake
69°00'
47°00'
SCALE IN MILES
1 0 1 2 3 4 5 6
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
RANKIN RAPIDS DEVELOPMENT
RESERVOIR MAP
International Passamaquoddy Engineering Board
JULY 1959
Dwg. No. TG7-099

PLATE 4-16


MATCH LINE
MAXIMUM POWER POOL
ELEVATION 910 FEET M.S.L.
CANADA
UNITED STATES
MATCH LINE
BIG BLACK RIVER
69°30'
47°00'
LITTLE BLACK RIVER
SAINT JOHN RIVER
BIG RAPIDS DAM
CUNLIFFE BROOK DIKE
UNITED STATES
SAINT JOHN RIVER
UNITED STATES
CANADA
RIVER
MAXIMUM POWER POOL
ELEVATION 630 FEET M.S.L.
ST. FRANCIS
ALLAGASH RIVER
ALLAGASH RIVER
SAINT JOHN RIVER
LINCOLN SCHOOL DAM
Long Lake
Round Pond
Umsaskis Lake
69°00'
47°00'
SCALE IN MILES
1 0 1 2 3 4 5 6
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
LINCOLN SCHOOL&BIG RAPIDS
RESERVOIR MAP
International Passamaquoddy Engineering Board
JULY 1959
Dwg. No. TG7-100

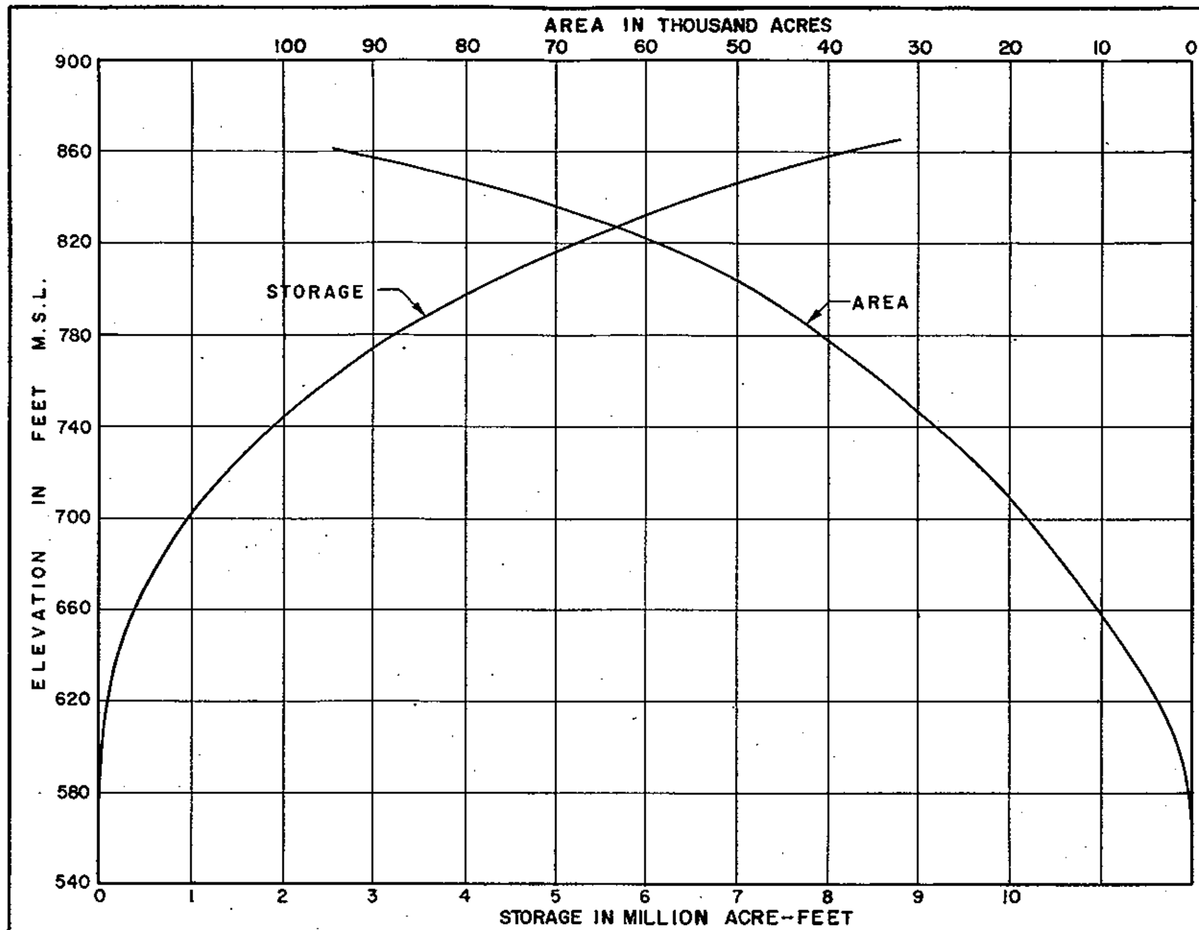


FIG.1 — RANKIN RAPIDS

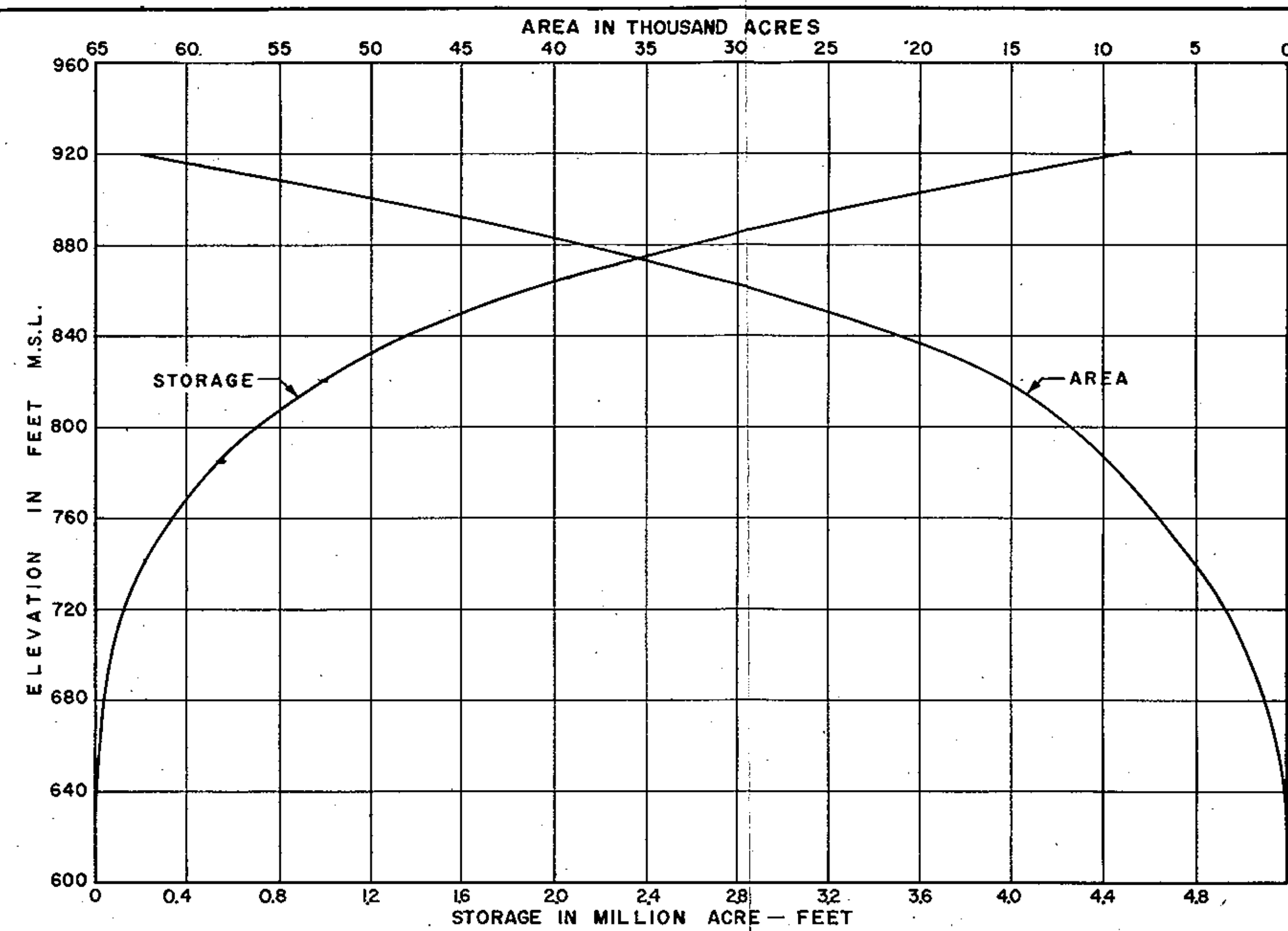


FIG.2 — BIG RAPIDS

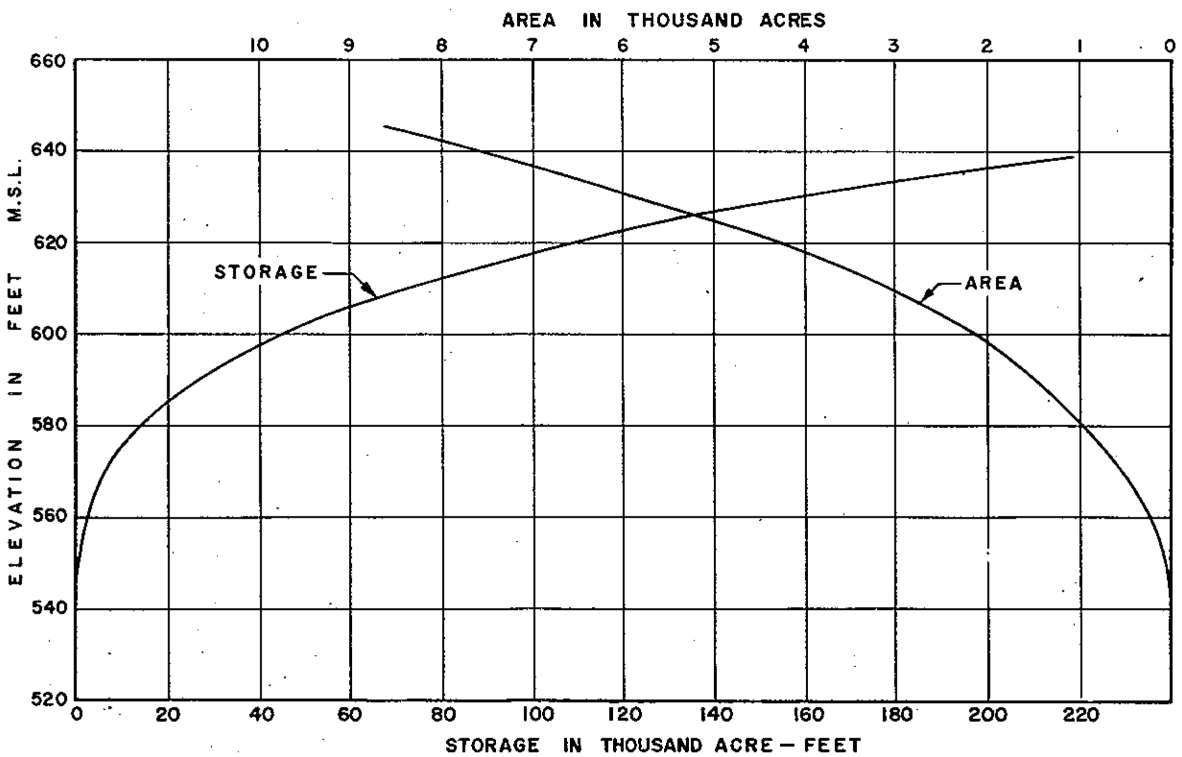


FIG.3 — LINCOLN SCHOOL

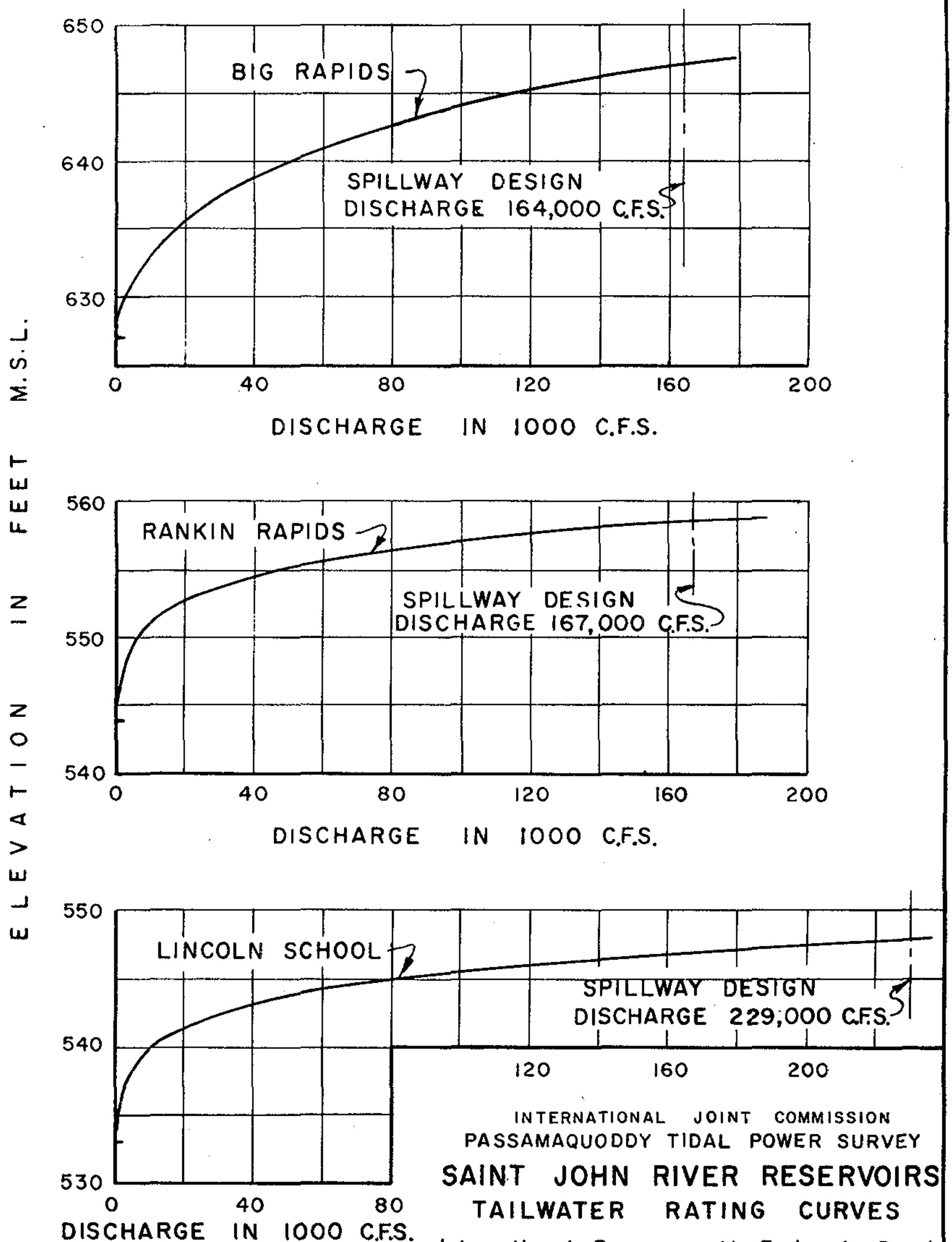
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

SAINT JOHN RIVER RESERVOIRS

AREA & CAPACITY CURVES

International Passamaquoddy Engineering Board

JULY 1959 Dwg. No. TG7-101



INTERNATIONAL JOINT COMMISSION

PASSAMAQUODDY TIDAL POWER SURVEY

SAINT JOHN RIVER RESERVOIRS

TAILWATER RATING CURVES

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JULY 1959

Dwg. No. TG7-102

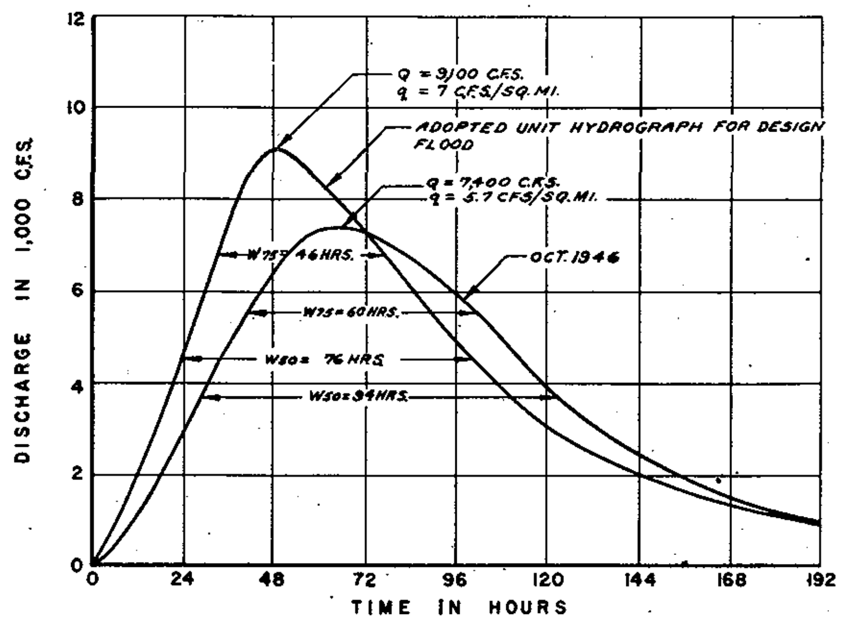

FIG. 1 - SAINT JOHN RIVER UPPER AREA 12 HR. UNIT HYDROGRAPHS
(ABOVE NINEMILE BRIDGE)

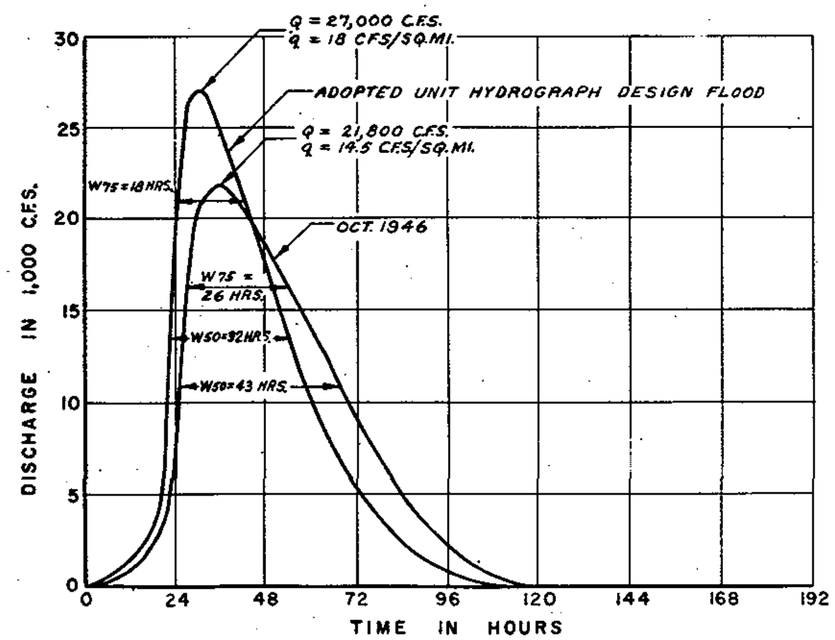

FIG. 2 - SAINT JOHN RIVER LOWER AREA 12 HR. UNIT HYDROGRAPHS
(BETWEEN NINEMILE BRIDGE AND RANKIN RAPIDS)

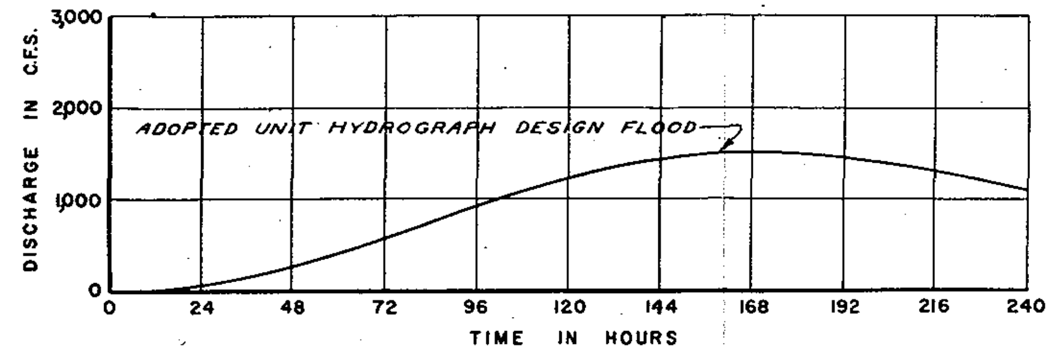

FIG. 3 - ALLAGASH RIVER UPPER AREA 12 HR. UNIT HYDROGRAPH

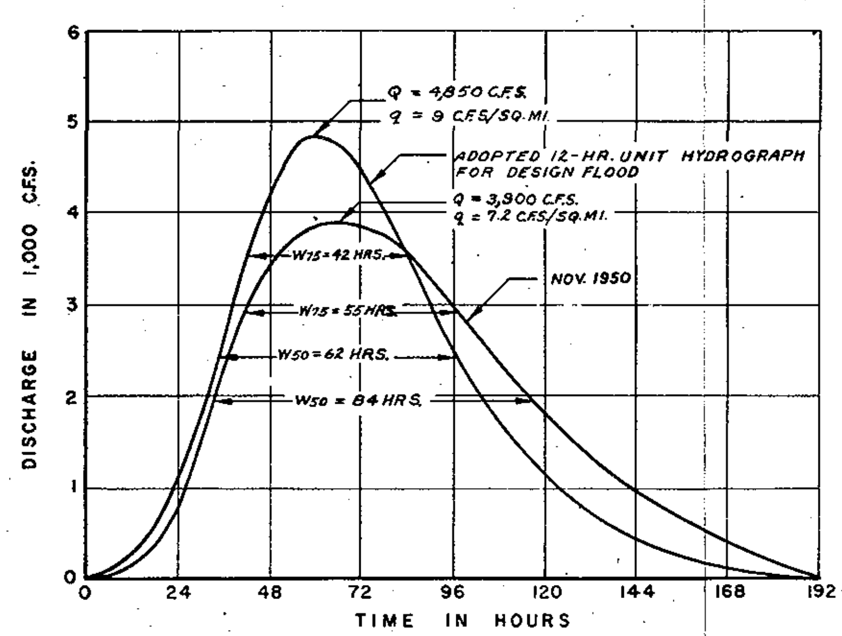

FIG. 4 - ALLAGASH RIVER LOWER AREA 12 HR. UNIT HYDROGRAPHS

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

UPPER SAINT JOHN RIVER

UNIT HYDROGRAPHS

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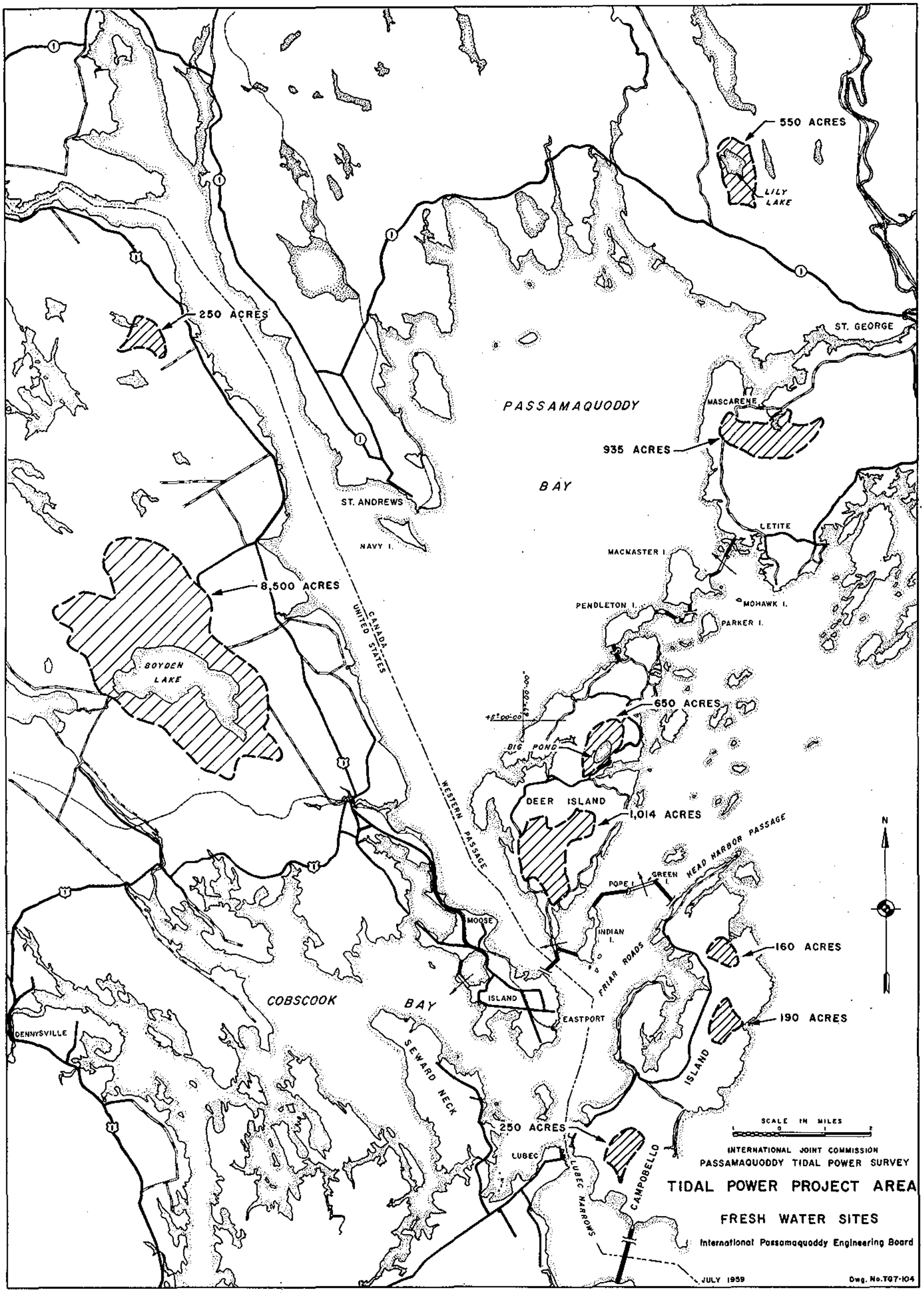

550 ACRES
LILY LAKE
250 ACRES
ST. GEORGE
PASSAMAQUODDY
MASCARENE
935 ACRES
BAY
ST. ANDREWS
NAVY I.
LETITE
MACMASTER I.
8,500 ACRES
PENDLETON I.
MOHAWK I.
PARKER I.
CANADA
UNITED STATES
BOYDEN LAKE
650 ACRES
BIG POND
WESTERN PASSAGE
DEER ISLAND
1,014 ACRES
HEAD HARBOR PASSAGE
POPE I.
GREEN I.
MOOSE
INDIAN I.
160 ACRES
FRIAR ROADS
COBSCOOK
BAY
ISLAND
EASTPORT
190 ACRES
DENNYSVILLE
SEWARD NECK
ISLAND
250 ACRES
LUBEC
LUBEC NARROWS
CAMPOBELLO
SCALE IN MILES
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
TIDAL POWER PROJECT AREA
FRESH WATER SITES
International Passamaquoddy Engineering Board
JULY 1959
Dwg. No.TG7-104

REPORT TO

INTERNATIONAL JOINT COMMISSION

ON

INVESTIGATION OF INTERNATIONAL

PASSAMAQUODDY TIDAL POWER PROJECT

APPENDIX 5

SELECTION OF PLAN OF DEVELOPMENT

BY

INTERNATIONAL PASSAMAQUODDY

ENGINEERING BOARD

OTTAWA, ONTARIO
WASHINGTON, D. C.

OCTOBER 1959

INTERNATIONAL PASSAMAQUODDY
ENGINEERING BOARD

INVESTIGATION OF INTERNATIONAL
PASSAMAQUODDY TIDAL POWER PROJECT

APPENDIX 5

SELECTION OF PLAN OF DEVELOPMENT

CONTENTS

CONTENTS - Continued

CONTENTS - Continued

APPENDIX 5

LIST OF TABLES

APPENDIX 5

LIST OF PLATES

INTERNATIONAL PASSAMAQUODDY
ENGINEERING BOARD

INVESTIGATION OF INTERNATIONAL
PASSAMAQUODDY TIDAL POWER PROJECT

APPENDIX 5

SELECTION OF PLAN OF DEVELOPMENT

5-01 PURPOSE

This appendix describes the studies to determine the most favorable plan for the development of tidal power in the international Passamaquoddy and Cobscook Bay area.

5-02 SCOPE

A large number of potential project layouts were compared to determine the one best suited for the site and the region. This appendix describes possible methods of pool arrangement and operation for power, the basis and criteria of comparative studies, comparisons of single-pool and two-pool plans leading to a more detailed study of two-pool arrangements, comparative layouts and estimates of the most significant two-pool plans, and the selection of the project layout for further detailed study. A simple two-pool plan with a powerhouse located between the high pool and the low pool was selected for design studies and cost estimates. The high pool would be in Passamaquoddy Bay and the low pool in Cobscook Bay. A map of the project vicinity is shown on plate 5-1.

Studies of the selected plan produced more refined field data and revised designs and estimates which were not available in the comparative studies. However, no other arrangement of the project would produce tidal power at a significantly lower cost than with the plan selected for specific study. If the tidal power project is authorized for construction, further study should be made of such other plans as discharging the powerhouse outflow alternately to the ocean and the lower pool, as well as design features such as the possible use of generating units of the type planned for the Rance tidal-power project in France.

5-03 POTENTIAL METHODS OF OPERATION

a. General. There are a number of basic methods by which hydroelectric power may be generated from pools filled or emptied by ocean tides. Many of these, including plans for the tidal power project at the Rance estuary in France, have been covered in technical literature. The most important and representative of the various methods of tidal power operation, without regard to their suitability to the site or the region, are shown on plates 5-2 and 5-3. These plates show schematic layouts, pool and ocean elevations and operating periods for the turbines and appurtenant gates together with typical power output curves. These are shown for an 18.1-foot tide range, the average in the Cobscook-Passamaquoddy Bay area. For the single-pool power studies, the pool area has been assumed to be 142 square miles, which is the combined pool area for the selected simple two-pool plan to be described later.

b. Single-Pool Projects.

(1) Single High Pool. As indicated in figure 1, plate 5-2, the single high pool would be refilled during the time that the ocean tide level is higher than the pool level. Power would be generated during the periods when the ocean level is lower than the pool level by discharging water through turbines from the pool to the ocean. The pool would be filled through gates and possibly through turbine water passages. No power would be generated during filling, nor would generation start until some time after filling has been accomplished. This delay in starting would be due to the need for a significant head for generating power (assumed as 6 feet). The need for significant head for power generation would also make it necessary to cease generating some time before pool filling is resumed. The dashed lines (figure 1, plate 5-2) indicate pool elevations and power outputs for a single high pool assuming the pool filling to be supplemented by pumping. Energy would be used by the tidal project during this pumping cycle, but the amount would be more than offset by the increase in energy generation made possible. Turbine-generator sets which could also be operated as motor-pump units would be used both for power generation and pumping. This type of unit is discussed in appendix 8, "Tidal Power Plant." With or without pumping, power generation would be intermittent with one period of no generation during each 12.42-hour tide cycle. During the pumping cycle, power would be absorbed from the system.

(2) Single Low Pool. The single-low-pool operation, figure 2, plate 5-2, would be the same as described above for a single high pool except that the flow would be reversed and power would be generated by flow from the ocean to the pool. Except for a minor difference caused by reduction in pool area at the reduced pool levels, energy output from a single low pool would equal that from a single high pool with equivalent turbine and gate capacities. Pumping would be applicable to this method of operation also. Power generation would be interrupted in each low tide period.

(3) Single Mean Pool. In this type of operation, figure 3, plate 5-2, power would be generated by flow from the ocean to the pool during periods of high ocean level, and then by flow from the pool to the ocean during the following period of low ocean level. Turbines which could generate power from flow in either direction would be used to reduce the cost of this method of tidal development. Comparing figure 3 for single mean pool with either figure 2 (single low pool) or figure 1 (single high pool) all on plate 5-2, shows that the maximum head for the single mean pool would be considerably less than for the other single-pool operations. Peak output would be reduced proportionately for like installations. Energy output would not be reduced as much as peak output as the single-mean-pool method would permit generation during a greater part of the tidal cycle.

(4) Single Mean Pool with Auxiliary Gates Added. Auxiliary gates when used with a single mean pool would provide an increase in energy by increasing the rate of pool filling and emptying thereby increasing the average head on the turbines during the following generating cycle (dashed line, figure 3, plate 5-2). Studies indicated that best utilization of the gates is obtained by opening them at about high tide and low tide for the respective filling and emptying cycles. As reversible turbines would be provided, this type of project could also be operated as a single-high or single-low pool when either is an advantage.

(5) Single Mean Pool with Auxiliary Gates and Reversible Pump-Turbines. As with a single-high or single-low pool, an energy increase might be obtained by the use of pumping just after gate closure. This type of operation is shown on figure 1, plate 5-3. Pumping becomes more advantageous as tide ranges increase because the minimum head necessary for generating, say 6 feet, would be secured with a shorter pumping period which would both decrease pumping energy and increase generating time.

c. Double Single-Pool Project.

(1) With a Common Powerhouse. Part of the disadvantage of a single-pool project lies in the time during each tide cycle when the power plant cannot operate because of insufficient head. This disadvantage could be overcome in part by operating a high pool and a separate low pool through a common powerhouse. Such a project is diagrammed on figure 2, plate 5-3. As shown on the diagram, gates would be provided between the powerhouse and each pool to permit power generation by controlled flow either from the upper pool to the ocean or from the ocean to the lower pool. Separate gates also would be required for filling the high pool and emptying the low pool. In this type of project, power interruptions would occur at times of change-over from one pool to the other. The interruptions would, however, be shorter than with a single pool.

(2) With Separate Powerhouses and Reversible Pump-Turbines. A possibility suggested by SOGREAH (Societe Grenoblois D'Etudes et D'Applications Hydrauliques) of France during discussions of the French bulb-type turbines was the use of a single high pool and a separate but electrically interconnected single low pool. Pumping at either pool would be accomplished using energy generated at the other. This arrangement would permit continuous power generation. A powerhouse would be required, however, at each pool.

d. Two-Pool Projects.

(1) Simple Two-Pool Project. A simple two-pool project, indicated in figure 3, plate 5-3, would consist of a high pool which would be filled through gates when high tide levels permit, and a low pool which would be emptied through another set of gates when tide levels fall sufficiently low. A powerhouse between the two pools would generate power continuously but at varying rates. The filling gates and emptying gates would be opened and closed when the head on the gates is nearly balanced; consequently the gate hoists would thus use minimum power.

(2) With Alternate Discharge to the Ocean. Powerhouse flow for a simple two-pool project tends to raise the level of the lower pool and to lower that of the upper pool. Both trends reduce the powerhouse head and consequently the energy developed. Additional head and energy could be secured by reducing the period during which the upper pool is drawn down or the lower pool is raised. This could be accomplished in the lower pool by the method shown on figure 4, plate 5-3. Two sets of gates in the

tailrace would be operated to direct the powerhouse outflow to either the ocean or the lower pool. From the time the lower pool emptying gates are opened, the project would operate as a single-high-pool project with the powerhouse discharging directly to the ocean until the minimum operating head is reached when powerhouse discharge would be redirected to the lower pool. The two sets of tailrace gates would be in addition to the filling gates and emptying gates.

5-04 LA RANCE PROJECT, FRANCE

Of the many sites studied for tidal power on the north coast of France, such as Aber-Vrach, the Bay of Mont-Saint-Michel and others, the most important is the Rance River estuary in Brittany. To date, an access road, an office building, a concrete laboratory and other buildings have been constructed. Construction on the project proper has been deferred pending further studies to reduce project cost.

The project would use one pool, 8.5 square miles in area, to develop power from the tides which have an average range of 27.8 feet at the site. The project would be operated as a single high, low, or mean pool as required to meet peak loads. These various methods of operation are made possible by use of a recently developed bulb-type power unit which can serve as a turbine, a pump, and a conduit, with flow in either direction. The axis of the turbine is horizontal and the water passages on both sides of the turbine are parallel to the axis. A small-diameter generator is inclosed in a "bulb" immersed in the water passage and in line with the turbine runner. Wicket gates are used and the runner blades are adjustable as in a Kaplan turbine. The turbines have a diameter of about 19 feet, and rotate at 88.2 r.p.m. The 32 generating units of the Rance project, according to the most recent information, are rated at 9,000 kw. each and have a total capacity of 288,000 kw. Sluiceways at one end of the powerhouse supplement the filling and emptying cycle. The average annual generation of 627 million kw.-hr. would be fed into the French power grid. Earlier reports indicated that 38 units were considered at one time and that the annual generation would be 800 million kw.-hr. from the 342,000 kw. of capacity.

The bulb-type turbines were considered for the Passamaquoddy site during the current survey and appeared about equal in power output to the best of the conventional turbines. They were not used in the powerhouse design, however, because of their high cost, low rotative inertia, untried design, and meager maintenance experience.

5-05 POWER STUDIES

a. General. Power determinations used in the general arrangement studies were made both manually and by the use of a high speed digital computer. In both methods, sea water at 64 pounds per cubic foot was routed from the ocean and back to the ocean through the project components, such as gates, turbines, and pools. The manual method was used for all single-pool studies and for preliminary studies of two-pool plans until a program for the computer was developed. The computer was programmed for simple two-pool plans only and was used to determine precisely variations in output between alternate plans. Since the computer was not programmed for alternate discharge to the ocean, the manual method was used for those studies. In the manual method, the power determinations were made for a few representative tide ranges and extended to obtain annual power production by use of tide range duration curves. It was determined that, within the range of installed capacities considered, power obtained from a mean tide range of 18.1 feet approximates the mean power computed from all tide ranges. Accordingly, most of the single-pool power determinations were obtained from a routing for a single 18.1-foot tide cycle.

The digital computer was programmed to determine pool elevations, ocean elevation, turbine head, and power output for the normal two-pool project at 15-minute routing intervals using the predicted tidal curve for a period of one lunar month. The month selected (October 1937) has a predicted mean tide range of 18.1 feet which is equal to the 19-year mean tide range as determined from observations at the Eastport, Maine tidal gauge. The computer study was programmed to permit an easy change of such characteristics as pool capacities, discharge characteristics and number of gates, and efficiency, discharge and number of turbines and generators. This permitted accurate determination of the output of any particular two-pool layout.

A detailed discussion of power computations is presented in appendix 13, "Project Power."

b. Turbines.

(1) Single-Pool Projects. Turbines for the single-high and single-low pool layouts, without pumping, could be of conventional design with flow through the powerhouse in one direction only. The single-mean-pool project (also double single-pool projects with a common powerhouse) would require turbines with two-way flow capability. The only known large

scale development where turbines with two-way flow are being considered is the tidal power project which is planned in the Rance River Estuary in Brittany, France. Turbines developed for this project would be of the horizontal-axis bulb type with about 9,000 kw. rating as previously described. Rotative speed is 88.2 r.p.m. and wheel diameter is about 19 feet. These turbines have been designed for both pumping and power generation with flow in either direction and may be used as orifices when desired. Development of these turbines was under the auspices of SOGREAH of Grenoble, France. SOGREAH was contacted early in 1957 to obtain characteristics of bulb-type turbines (of 320-inch diameter and 40 r.p.m.) suitable for the Passamaquoddy tidal power project, but data was not received in time for the general single-pool power studies. Accordingly, the energy for single-pool layouts was computed using preliminary Kaplan turbine characteristics modified by judgment to account for the pumping cycle and two-way flow. When limited data was received in June 1958 from SOGREAH, it was reviewed to check the validity of the computations already made on the basis of assumed characteristics. It was found that the SOGREAH turbines would generate about 10 percent more energy than shown by the computations using preliminary turbine characteristics. However, later information on Kaplan and fixed-blade turbines also showed a similar increase in energy over the computations based on preliminary turbine characteristics. On this basis, the computations based on the preliminary turbine characteristics gave an accurate comparison of the types of projects studied.

(2) Two-Pool Projects. Turbine characteristics used for analysis of the two-pool projects presented in this appendix were based on preliminary curves for fixed-blade, propeller-type turbines of 320-inch diameter, and turning at 40 r.p.m.

c. Generators and Motors. In the general arrangement studies, generators and motor-generators were assumed of sufficient capacity so that output of the turbines or pump-turbines would not be restricted by electrical limitations. Generator and motor efficiencies were assumed to be 97 percent for all conditions.

d. Losses. Hydraulic losses depending on the layout under consideration, would occur in the gate channels and gate structures and powerhouse approach and tailrace channels. The effects of these losses and powerhouse intake losses were evaluated and included in the annual energy outputs of the project considered. Losses due to leakage through the dams were not evaluated in the comparative studies.

e. Single-Pool Power Evaluations. Energy outputs for various types of single-pool projects were computed, as previously described, from the 18.1-foot mean tide range. The Kaplan turbines were assumed to operate at maximum power at higher heads, and with the combination of gate and blade angle producing 70 percent efficiency at lower heads. Pumping efficiency was assumed 70 percent for all heads for the studies involving this feature. Plate 5-4 shows the results of these studies and a comparison of energy outputs for two-pool and single-pool projects having the same total pool area (based on Passamaquoddy tides).

The comparison shows that a two-pool project with less than 60 turbines would yield more energy than a single-pool project with the same number of turbines. The reasons for this can be illustrated by assuming only one turbine to be installed for both a single-pool layout and a two-pool layout. During a tide cycle, the turbine with the two-pool plan could operate continuously at a head nearly equal to the full tide range because the one turbine would not have sufficient discharge capacity to change appreciably the level of either pool. The one turbine operating at a single-high or low-pool project, where the head would be from pool to ocean elevation, would operate with a mean head of about one-half the tide range or a half of that at the two-pool project. Energy output would be decreased accordingly. Similar reasoning applies to the single-mean-pool type project since the pool would be at about mean sea level and the head would vary between the maximum prevailing at high or low tide and zero. In addition, power generation would cease at any single-pool project some time during the tide cycle because of insufficient head. In the above illustration, pumping would not change the picture materially since the pumping discharge capability of one pump-turbine could not affect the pool elevations significantly. As the installed capacity is increased, however, the energy per turbine for the two-pool scheme would decrease at a faster rate than for the single-pool schemes. This is true because additional turbine discharge causes a greater reduction in turbine head in the two-pool plan than in the single-pool projects since turbine discharge not only lowers the high pool stage but raises the low pool stage. With about 60 or more power units, more energy could be secured with some sort of a single-pool plan than with a two-pool plan.

f. Two-Pool Power Evaluations. The power outputs for the simple two-pool plans specifically described in this report (not including study 1-1.211, plate 5-14) were determined by electronic computer runs which included the essential characteristics of the layouts such as the area-capacity curves of the pools (given in appendix 4, "Basic Hydrologic Data"), the number and varying discharge of gates, and the number of turbines to be installed. The fixed-blade turbines were assumed to operate for best efficiency at all heads, and the number in operation during neap tides was reduced as necessary so that turbine head would not become less than about 6 feet. The above described power studies were preliminary, and were directed solely toward determining the most suitable project arrangement. More detailed power studies were made for the finally adopted project layout, and these are described in appendix 13, "Project Power." The two-pool power studies described above and others, both manual and machine, were reduced to the curve form shown on plate 5-5 which was used to estimate power outputs of other layouts. This plate shows annual energy per turbine plotted against gross pool area per turbine for two-pool developments for several ratios of the smaller pool area to the larger pool area. Gross pool area is the sum of both pool areas. Pool areas are those at el. +6 and -6 mean sea level for the high and low pools respectively. The energy output shown by the curves was adjusted to reflect losses due to the channels and gates in the particular two-pool project layout.

5-06 COMPARISON OF SINGLE-POOL PROJECTS WITH TWO-POOL PROJECTS

a. Annual Energy per Unit of Capacity. It is apparent from plate 5-4 that a single-pool project at Passamaquoddy would require more generating units to produce the same amount of energy as a two-pool project with the same gross pool area, unless the installation is larger than about 60 units of the assumed size. The relatively small yield of one-pool plans is illustrated further by the following comparisons:

(1) The Rance River tidal power project planned in Brittany, France, when built may be operated as a single-mean-pool project, or as a single-high or a single-low-pool project. The installation will include 32-9000 kw. bulb-type pump-turbines and will generate 627 million kw.-hr. per year. This amounts to 2,180 kw.-hr. per year per kilowatt of installed capacity, including the advantage of pumping. This means that each kilowatt of capacity will be used at an average of about 25 percent of its rating. For the 38-unit project with an 800 million kw.-hr. annual energy, the percentage would be 27 percent.

While the project is said to be capable of operating at any time during the tide cycle to meet peak loads, generation will necessarily be stopped on each rising tide and falling tide when the head is insufficient.

(2) In 1935-37, the U.S. Army Corps of Engineers considered a single-high-pool tidal power project using Cobscook Bay in Maine, with a total installed capacity of 150,000 kw. Average annual generation was estimated as 340 million kw.-hr., or 2,260 kw.-hr. per year per kilowatt of capacity. Average utilization per installed kilowatt would be about 26 percent. Operation would also be intermittent.

(3) The recommended two-pool project would generate about 1,800 million kw.-hr. a year with 30 - 10,000 kilowatt generators. The generation would be more than 6,000 kw.-hr. per year per kilowatt of capacity. Average utilization per installed kilowatt would be about 69 percent. Operation would, in this case, be continuous, with some capacity available at all times.

b. Power Cost Comparison. The previously cited plate 5-4 shows that a single-high-pool project with 50 power units and 100 filling gates, without pumping, would generate about 1,900 million kw.-hr. of energy per year or about as much as a 30-unit two-pool project with an equal total pool area. The latter is equivalent to the recommended plan which is described later in this report. It would have 60 more gates, one more dam and one more lock than the single-pool project, but 20 fewer power units. The power units are a relatively large part of the cost, and consequently the single-pool project would cost about $50,000,000 more than the two-pool project, with a power cost 17 percent greater than for the two-pool project. Other unfavorable aspects of the single-pool project include intermittent generation, and the 60 to 70 percent greater peak generation for the same amount of energy. A further increase from 50 to 100 units for the single-pool project would increase annual energy by only 1,000 million kw.-hr. at a cost of about $215,000,000 not including excavation and cofferdams. The cost of this added energy, $0.215 per annual kilowatt-hour in powerhouse costs alone, is excessive and illustrates the impracticability of increasing the size of a single-pool project. The possibility of smaller single-pool projects using Passamaquoddy Bay alone, or Cobscook Bay alone, was also examined. These would permit use of Carryingplace Cove as the powerhouse site as in the recommended plan. Comparative studies of two such single-pool plans (Passamaquoddy Bay with 35 generating units and Cobscook Bay with 5 or 15 units) showed that the cost of energy would be about 17 percent more than for the recommended two-pool plan, not taking into account the intermittent generation inherent to a single-pool project.

c. Double Single-Pool Project with a Common Powerhouse. The fact that a single-high-pool project would generate about half the time, as would a single-low-pool project, led to the consideration of a combined single-high-pool project and single-low-pool project in which the same generating units would serve both pools. This arrangement would make it possible to use the powerhouse most of the time and thus overcome one disadvantage of single-pool operation. A preliminary power study indicated that 30 generating units would produce 7 percent more energy than the same number of units in a simple two-pool project using the same pools. This increase was not sufficient to overcome the loss of nearly 100,000 kilowatts of dependable capacity and the additional cost of construction and operation. Inherent disadvantages would include the need for about 80 additional gates to permit connecting the powerhouse to both pools, increased complexity of operation, and probably a less favorable powerhouse location. Loss of dependable capacity would result because generation would be stopped twice during each tide cycle when operation would be changed from a high-pool to a low-pool method, and back again. These operations would involve opening one set of gates, closing another and changing the direction of flow through the turbines. The latter would involve stopping the turbines, and starting them turning in the opposite direction. Site studies of this method of operation were not undertaken because of the unfavorable factors.

d. Two Single-Pool Projects with Separate Powerhouses. In this method of interconnected single-pool projects, described in paragraph 5-03c(2), the cost and energy output would be about the same as for two separate one-pool projects. Although the combination could generate power continuously, the average energy output per turbine would be relatively small as each plant would operate as a single-pool project. Site studies of this method of operation were not undertaken because the cost of the tidal energy would be relatively great.

e. Conclusions. Physical characteristics of the tidal power project, such as the low head available, necessitate the handling of large volumes of water requiring costly turbines and gate structures. These characteristics make it unlikely that the tidal power project could be developed to the stage where the cost of tidal power would be substantially less than from the cheapest alternate source. Thus the problem was reduced to determining the project layout that would produce the cheapest power. Because the use of single-pool projects would, as developed in the previous subparagraphs, (1) increase the cost of power, (2) increase peak generation rates, and (3) generate power intermittently, further consideration of single-pool arrangements including refinement of power studies was abandoned in favor of two-pool projects.

5-07 COMPARATIVE STUDIES OF TWO-POOL PROJECTS

a. General. There are a number of possible sites and arrangements of the several dams, the powerhouse, gates, and locks which would be required for a two-pool tidal power project. Comparative studies were made early in the investigation to direct foundation exploration in the interests of economy to those sites which could be shown to be most favorable on the basis of the data available at the time and power studies. It was found necessary to compare arrangements of the entire project instead of separately comparing sites for each dam or structure. To compare various integrated plans and select one for detailed study of dams, structures, power output and cost, it was necessary to use preliminary designs for project components, or designs taken from earlier studies of the tidal project. Thus the comparative studies discussed in this appendix are not based on the designs developed in the detailed study and discussed in the other appendices of this report. Many possible arrangements for a two-pool project were studied, all of which can be placed in three groups depending upon whether Passamaquoddy Bay would be the upper pool, the lower pool, or divided between the two pools. Other basic variations include location of the tidal dams for the pool including Cobscook Bay, and the possibility of diverting the powerhouse outflow directly to the ocean, by-passing the lower pool during a part of each tide cycle. The most economical arrangements developed for each of the above variations in the initial phase of the comparative studies are discussed and compared in subparagraph d. On the basis of these comparisons, two arrangements involving Passamaquoddy Bay as the upper pool (studies 4-6.22 and 7B-7.22) were modified and refined and the results reported as in studies 4-6.33 and 7B-7.212. A new study was also made with Passamaquoddy Bay in the lower pool (study 7B-7.512). This last series of three studies differed from the initial phase in that variations in the number of filling and emptying gates as well as in the number of generating units were included. The discussion of the final series of three studies is presented in subparagraph e. Discussion of some of the factors which pertain to the development of the project layout follows in subparagraph f.

b. Basis of Comparison. The two-pool plans have been compared on the basis of a ratio which is called, for this report, the comparative index. This ratio is the total cost of the major construction features of the tidal project (comparative cost) divided by the average annual energy which could be generated by the project layout under consideration. A small comparative index is favorable. In estimating the costs of the various layouts, overhead, engineering and contingency items usually regarded as being proportional to construction costs were omitted. The use of only the average annual energy in the denominator of the ratio, comparative index, may seem to imply that the index neglects capacity benefits entirely and consequently that the index, while useful, does not take the very important factor, capacity, into account. However, average annual energy represents comparative power benefits including capacity, because for a two-pool project, capacity benefits are in about the same ratio to the total power benefits for any of the layouts considered.

c. Basis of Comparative Estimates.

(1) General. Since studies of various project arrangements were necessarily made early in the survey before new basic data was available, the considerable information available from previous investigations of tidal power projects in Passamaquoddy and Cobscook Bays was used including topographic and hydrographic mapping, exploratory drilling, materials investigations, and designs for various structures and dams. The most complete data covered the sites for structures and dams in the vicinity of Carryingplace Cove, Estes Head, Treat Island and Lubec. At other possible sites, the topographic, hydrographic and geologic data available from various sources were sufficient to support comparative estimates. The comparative estimates also showed which of the sites were suitable for additional underwater mapping and foundation drilling. During the period of underwater mapping and foundation exploration, comparative studies were continued to permit timely selection of a specific arrangement for which a detailed study and estimate could proceed. Since subsequent mapping and foundation analysis disclosed conditions broadly consistent with the comparative estimates, revision of these estimates has not been justified. The new data, however, have been incorporated in the design and final estimate for the selected plan. The costs used in the comparative studies were preliminary and used solely for selecting the most suitable project arrangement. They are not the same as the costs in appendix 17, "Estimates of Cost," which are the basis for evaluating the economic justification of the tidal power project.

(2) Unit Prices. Unit prices were based on preliminary estimates of construction costs in the Passamaquoddy Bay area, using labor costs applicable in the United States. Unit prices for various items were as shown in table 5-1. The same prices were used in each of the comparative studies except that a suitable method of handling and transporting excavated or borrowed materials and the applicable price were selected in each case. These were preliminary prices for comparative purposes and are not the same as the prices developed in later detailed study and shown in appendix 17, "Estimates of Cost."

(3) Dams. In all comparative studies, the dams were rockfill with an earthfill blanket and riprap. A typical cross section of the dams between the ocean and either pool is shown on plate 5-6. The rockfill above el. -30 was assumed to be derrick stone of a size large enough to stand on a slope of 1 (vertical) on 1.75 (horizontal) when placed in the tidal currents flowing over the partly completed fill. For dams between the upper pool and the lower pool, the cross section was assumed to be similar except that, in place of derrick stone, random rockfill with a 10-foot facing of large durable rock would be used above el. -30. It was assumed that derrick stone, the durable facing and all riprap for the dams and cofferdams would be obtained from borrow. In computing the volume of fill in the dams, allowance was made for settlement or displacement of the foundation materials on the basis of a preliminary evaluation of the physical characteristics of the foundation sediments. Such an allowance was made for the following dams:

Estes Head to Treat Island and Dudley Island
Dudley Island to Lubec
Dudley Island to Campobello Island
St. Andrews to Deer Island

All cofferdams were assumed to be of the cross section also shown on plate 5-6. These would consist of rockfill with an earthfill blanket and riprap. The design of dams and cofferdams was later studied and revised for the detailed estimate of the selected plan of development as shown in appendix 9, "Tidal Dams and Cofferdams."

(4) Navigation Locks. The navigation locks were assumed to be large enough to pass current traffic with a moderate increase in the maximum sizes of vessels. The clear dimensions of locks in Head Harbour Passage and Western Passage were assumed

to be 415 feet by 60 feet with a depth of 21 feet at mean low water. For smaller locks in Letite Passage and in Quoddy Roads the clear dimensions are 95 feet by 25 feet with a depth of 10 feet at mean low water. For locks at Treat Island and St. Andrews, the clear dimensions are 370 feet by 50 feet with a depth of 19 feet at mean low water. Where the elevation of bedrock at the proposed site is high enough, the lock walls were assumed to be of rock with an anchored lining of reinforced concrete. At other sites, more expensive concrete walls of the gravity type were estimated. The design of the locks and the draft of the smaller locks were later studied and revised for the detailed estimate of the selected plan of development as shown in appendix 7, "Navigation Locks." Locks of the size built in the St. Lawrence seaway could be provided in any of the arrangements shown, if warranted by future traffic to Passamaquoddy Bay.

(5) Filling and Emptying Gates. Filling gates and emptying gates were assumed to be the vertical-lift type set in a submerged water passage of venturi shape. The comparative estimates of quantities for a gate unit, including operating machinery and structures, were based on the drawings and estimate sheets made in 1928 by Dexter P. Cooper, Inc. These gates, 30 x 30 feet in size, would be operated hydraulically by changing the submergence of a counterweight. Either a double-type setting (gates in pairs, one above another) or a single-type setting was used in the comparative estimates depending upon topography, hydrography, and foundation conditions at each site. Prior to the final series of comparative studies, 180 gates were used in all arrangements. In the final series the number of gates was adjusted to approximately the most economic number for that particular arrangement (160 gates in study 4-6.33 and 130 gates in studies 7B-7.212 and 7B-7.512). The double-type gate settings were not used in the final series of comparative studies because preliminary design studies showed them to be considerably more expensive than indicated by the 1928 information. In the layout studies with the most favorable comparative indexes, physical conditions at sites for gates were favorable for the single-type setting. The design of the filling gates and the emptying gates was later studied and revised for the detailed estimate of the selected plan of development as shown in appendix 6, "Filling and Emptying Gates."

(6) Powerhouse. In the comparative estimates, the powerhouse was estimated on the basis of layouts and quantity estimates made by the Corps of Engineers, U.S. Army, in 1935 - 1936. The unit spacing was changed from 83 to 86 feet, with structural contraction joints between each unit. Turbines

320 inches in diameter and revolving at 40 r.p.m. were assumed. Current estimated prices for generators, turbines and governors were obtained from manufacturers. The number of generating units was varied. Several of the arrangements reported herein (including study 4-6.33 in the final series) were compared using 20, 25, and 30 generating units. Studies 7B-7.212 and 7B-7.512 were compared using 15, 18, 20, 25, and 30 units. Forty-four units were compared with 30 units in the arrangement of study 1-1.211 and in several other arrangements. Some comparisons using 35 units were made by interpolating the cost of the power facilities. On the basis of these investigations, the number of units in each of the reported arrangements is approximately the number for minimum cost of power. The headrace and tailrace were designed for a maximum velocity of 3 feet per second with the water surface at the minimum operating level. This velocity was based on a preliminary economic analysis of power losses and excavation costs. The design of the powerhouse was later studied and revised for the detailed estimate of the selected plan of development as shown in appendix 8, "Tidal Power Plant and Corrosion Prevention."

(7) Remaining Construction Items. The only other item of construction included in the comparative estimates was the relocation of the road and railroad to Eastport. This included a bridge across the headrace for those arrangements with a powerhouse at Carryingplace Cove. An allowance of $1,000,000 for that item was made without an estimate of quantities since further refinement was unjustified at that stage of the investigation. Costs for the switchyard, transmission lines, service facilities, and land acquisition were not included because the proportional cost of these would not vary significantly between the plans considered.

(8) Use and Handling of Excavated Materials. It was assumed that earth and rock from required excavations would be suitable for use in dams and cofferdams except that derrick stone and riprap would be borrowed from quarries for large durable stone. Rock required to be removed from cofferdams was assumed to be available for reuse where no less expensive alternative material would be available. Where required earth excavation would be less and required rock excavation more than needed for the assumed cross-section of the dams, substitution of excess rock for earth in the blanket was allowed in quantities up to one-half the volume of the blanket. The source, destination,

and methods of handling and transporting all excavated material were considered in relation to cost. Excavation was wasted and replaced by borrow where this procedure would be less costly.

d. Initial Phase of Comparative Studies.

(1) General. The many comparative studies were screened first by eliminating those which had a markedly higher ratio of cost to power output. Of the remaining plans, each of the five discussed below is the most favorable in one of several categories. Thus the Pope Islet plan (study 4-6.22) is the most favorable of those with Passamaquoddy Bay in the upper pool. Study 4-6.52 is the most favorable with Passamaquoddy Bay in the lower pool. In the St. Andrews plan, Passamaquoddy Bay would be divided between the upper pool and the lower pool. Treat Island Plan 1 would not include Friar Roads and Quoddy Roads in the lower pool, thus substantially reducing the area of the pool and the energy output. The fifth plan would direct the powerhouse discharge through tailrace gates alternately to the ocean and the lower pool. These five plans are described in following paragraphs. Comparative data are shown in table 5-2.

(2) Study 4-6.22 - Pope Islet Plan. The upper pool in this arrangement would be in Passamaquoddy Bay and Western Passage as shown on plate 5-7. The lower pool would be in Cobscook Bay, part of Quoddy Roads, Friar Roads and part of Head Harbour Passage. Fifty filling gates would be provided in Letite Passage, and sixty at Deer Island Point to discharge into Western Passage. Seventy emptying gates would be at Head Harbour Passage on an underwater ridge between Pope Islet and Green Islet. The powerhouse would be at Carryingplace Cove off Moose Island. Navigation locks would be provided at Head Harbour Passage, Western Passage, Letite Passage and Quoddy Roads. This arrangement would produce 1,850 million kw.-hr. per year with 30 generating units. The comparative cost is $254,000,000. The comparative index is $0.137. A major advantage of this plan is that all of Passamaquoddy Bay and Western Passage would be in the upper pool. This would improve navigation and harbor depths in both Canada and the United States because the water surface in the upper pool would vary only from about el. +3 mean sea level to about 1 foot below present high tide. The controlling depth for navigation in the St. Croix River to Calais and St. Stephen would be 22 feet at mean low stage of the upper pool instead of the existing 7 feet

at mean low tide. The utility of existing waterfront facilities at Calais, St. Stephen and St. Andrews, would be enhanced by a lesser tide range and generally higher water levels. Waterfront facilities which may be built in the future would be less costly to construct and operate because of the lesser tide range in both pools. These benefits were not evaluated for the comparative studies. A disadvantage of this plan is that existing piers and wharves at Eastport and Lubec would be on the lower pool and would be less desirable because the water surface would remain below mean sea level at all times. This was not evaluated for the comparative estimates either.

(3) Study 4-6.52 - Passamaquoddy Bay in Lower Pool. This arrangement, shown on plate 5-8 is similar to that of study 4-6.22 except that Passamaquoddy Bay would be in the lower pool and Cobscook Bay would be in the upper pool. This arrangement would produce 1,970 million kw.-hr. per year with 30 generating units. The comparative cost is $259,500,000 and the comparative index $0.132. The utility of existing piers and wharves at Eastport and Lubec would be enhanced by the lesser tide range and generally higher water surface of the upper pool. Navigation to Calais and St. Stephen would be severely handicapped because the water level in the St. Croix River would remain below mean sea level at all times. Controlling navigation depths would be about 4 feet at extreme low stage, about 9 feet at mean low stage, and about 12 feet at average stage of the lower pool.

(4) Study 6A-2.613 - St. Andrews Plan. This arrangement, shown on plate 5-9, would divide Passamaquoddy Bay between the upper pool and the lower pool with a dam extending from St. Andrews to Deer Island. The part of the bay northeast of this dam would be the lower pool with an area of 51.6 square miles measured at el. -6 mean sea level. The upper pool would include the remainder of Passamaquoddy Bay, Western Passage, Friar Roads, and Cobscook Bay with a total area of 87.2 square miles at el. +6 mean sea level. Dams at the south end of Indian Island between the upper pool and the ocean were found to be more economical than dams at Pope Islet in this case. This is explained by the fact that the upper pool would be sufficiently large that extending the pool to Pope Islet would not produce enough additional energy to justify the more costly location. For a similar reason, the dam at Lubec Narrows, excluding Quoddy Roads from the upper pool, was found desirable. The powerhouse would be located on Navy Island near St. Andrews. One hundred filling gates would be provided at Indian Island. Eighty emptying gates would be

provided, 70 of which would be in Letite Passage and the remaining 10 in Little Letite Passage. Navigation locks would be provided between Head Harbour Passage and Western Passage, in Letite Passage, Lubec Narrows, and at Navy Island near St. Andrews. This plan would produce 1,880 million kw.-hr. per year with 30 units. The comparative cost is $255,800,000 and the comparative index is $0.136. This arrangement would produce 2 percent more energy than that of study 4-6.22 with a comparative index 0.7 percent lower. The western part of Passamaquoddy Bay, St. Andrews Harbour, all of Western Passage, and the lower St. Croix River to Calais and St. Stephen would be included in the upper pool with pool levels about the same as with study 4-6.22, and in addition, Cobscook Bay and the waterfront facilities at Eastport and part of Lubec would be in the upper pool. Those at St. George would be in the lower pool. Foundation conditions along the dam alignment extending across Passamaquoddy Bay from St. Andrews to Deer Island were being investigated at the time the estimates given in table 5-2 were being prepared. On the basis of visual examination of the first cores showing that the underwater foundation would be soft clay, an allowance was made for foundation settlement as previously mentioned in paragraph 5-06c(3). This settlement allowance is reflected in the estimate in table 5-2 which shows a comparative index with an advantage of $0.001, about 0.7 percent, over study 4-6.22. As the foundation investigation progressed, using three drill holes and observing the penetration of sonic fathometer waves into the underwater sediment, it became evident that the layer of soft clay varied in depth from about 29 feet near St. Andrews to about 62 feet under deeper water near Deer Island. Because the extent and characteristics of this material raised doubt that the settlement allowance was adequate, and because at best the St. Andrews plan would offer no significant economic advantage over study 4-6.22, further foundation explorations and layout studies of the St. Andrews plan were abandoned.

(5) Study 7B-7.22 - Treat Island Plan 1. The upper pool would be Passamaquoddy Bay and Western Passage in this arrangement (plate 5-10). The lower pool would be limited to Cobscook Bay by dams at Treat Island and Dudley Island. Sixty filling gates would be provided in Letite Passage and sixty at Deer Island Point to discharge into Western Passage. Sixty emptying gates would be located at the entrance to Cobscook Bay (40 on the north side of Treat Island and 20 on the south side). The powerhouse would be at Carryingplace Cove off Moose Island. Navigation locks would be provided at Western Passage, Letite Passage, and at the entrance to Cobscook Bay. This arrangement would produce

1,380 million kw.-hr. per year with 20 generating units. The comparative cost is $177,500,000, and the comparative index $0.129. This plan has the lowest comparative index of those studied in the initial phase and also has the advantages of study 4-6.22 with respect to the location of the upper pool in Passamaquoddy Bay and Western Passage. This plan was subsequently improved by the inclusion of Quoddy Roads in the lower pool, estimated with greater refinement and compared with the recommended plan in the final series of comparative studies.

(6) Study 1-1.211 - Alternate Discharge to the Ocean. This arrangement, shown on plate 5-11, is of the type proposed by Dexter P. Cooper, Inc., in 1928 for the international tidal power project. It is basically similar to study 7B-7.22 except that provision is made for bypassing the lower pool by diverting the powerhouse outflow directly to the ocean during a part of the tide cycle. This required the location of the Western Passage dam and filling gates at Johnson Cove on the east side of Moose Island in line with the powerhouse in Carryingplace Cove. Thirty tailrace gates would be provided at each side of the tailrace to direct the powerhouse flow alternately to the ocean and the lower pool. Of several plans in the initial series incorporating alternate discharge to the ocean, this plan has the lowest comparative index. This arrangement with 30 generating units would produce 1,800 million kw.-hr. a year. The comparative cost is $252,300,000, and the comparative index $0.140. In comparison with study 4-6.22, this arrangement has disadvantages in addition to the greater comparative index and slightly smaller power output. These disadvantages arise from the need for directing the entire powerhouse outflow alternately through one set of tailrace gates to the ocean and through another set of tailrace gates to the lower pool. This would entail operation of both sets of tailrace gates twice during each tide cycle or about 4 times a day. Closing the gates on the ocean side and opening the other gates would require close coordination to control the water surface in the tailrace at or slightly above tide level while the gates to the lower pool are being opened. The use of tailrace gates would increase the project power consumption and the cost for operation and maintenance, neither of which are accounted for in the comparative index.

(7) Summary. From the foregoing initial phase of the comparative studies, studies 4-6.22 and 7B-7.22 were selected for refinement and included in the final series of comparative studies. The new study numbers are 4-6.33 and 7B-7.212. Study 4-6.52 was not selected for further study because the disadvantages in navigation depths would outweigh the 4 percent lower cost of

power. However, investigation of the possible economic advantage of using Passamaquoddy Bay in the lower pool was continued by making study 7B-7.512 as the third study of the final series. Study 6A-2.613 was abandoned because of the poor underwater foundations found to exist between St. Andrews and Deer Island. Alternate discharge to the ocean was abandoned because the comparative indexes of study 1-1.211 and other studies incorporating this feature were greater than that of the recommended plan, and because increased operation and maintenance costs, not reflected in the comparative index, would tend to further increase the cost of power from this arrangement.

e. Final Series of Comparative Studies.

(1) General. Study 4-6.33 (the recommended plan), study 7B-7.212 and study 7B-7.512 are described and compared below. The first two plans would use Cobscook Bay as part of the lower pool; they differ principally in the location of the dams for this pool. In the recommended plan, the lower pool would extend to dams at Pope Islet and Green Islet. In the second plan the lower pool would be smaller, extending only to a dam at Treat and Dudley Islands. The third plan would be the reverse of the second, in that Cobscook Bay would be in the upper pool, and Passamaquoddy Bay in the lower pool. Pertinent data on costs and power output of these three arrangements are shown in table 5-3. The results of studies of the final series are comparable with each other but not exactly comparable with studies of the initial phase due to advances in design and power studies. The designs and costs in the final series of comparative studies were not the same as those developed later for the detailed estimate of the selected plan and shown in other appendices.

(2) Study 4-6.33 - Recommended Plan. Passamaquoddy Bay and Western Passage are the upper pool in this arrangement as shown on plate 5-12. The lower pool would include Cobscook Bay, part of Quoddy Roads, Friar Roads and part of Head Harbour Passage to the dam between Green Islet and Campobello Island. Quoddy Roads was included in the lower pool because this would reduce the comparative index and therefore the cost of power. Forty filling gates would be provided in Letite Passage, and fifty at Deer Island Point to discharge into Western Passage. Seventy emptying gates would be located between Pope Islet and Green Islet at Head Harbour Passage. The powerhouse would be located at Carryingplace Cove off Moose Island. Navigation

locks would be provided at Head Harbour Passage, Western Passage, Letite Passage and Quoddy Roads. This arrangement would produce 1,774 million kw.-hr. per year with 30 generating units. The comparative cost is $243,000,000 and the comparative index is $0.137.

(3) Study 7B-7.212 - Treat Island Plan 2. Passamaquoddy Bay and Western Passage would be the upper pool in this arrangement also (plate 5-13). The lower pool would include only Cobscook Bay and part of Quoddy Roads. As in the recommended plan, the inclusion of part of Quoddy Roads would be justified by the value of the additional energy generated which would be appreciably greater than the additional cost. Forty filling gates would be provided in Letite Passage and forty at Deer Island Point to discharge into Western Passage. Fifty emptying gates would be located at the entrance of Cobscook Bay (30 on the north side of Treat Island and 20 on the south side). The powerhouse would be located at Carryingplace Cove off Moose Island. Navigation locks would be provided at Western Passage, Letite Passage, Quoddy Roads and at the entrance to Cobscook Bay. This arrangement would produce 1,320 million kw.-hr. per year with 20 generating units. The comparative cost is $170,200,000 and the comparative index $0.129.

(4) Study 7B-7.512 - Treat Island Plan 3. This arrangement shown on plate 5-14 is similar to that of study 7B-7.212, described above, except that Passamaquoddy Bay would be in the lower pool and Cobscook Bay would be in the upper pool. This arrangement would produce 1.379 billion kilowatt-hours per year with 20 generating units. The comparative cost is $177,500,000 and the comparative index $0.129.

(5) Summary. The recommended plan, study 4-6.33, would produce 35 percent more energy than study 7B-7.212, but at a 6 percent greater comparative index. An appraisal of operating and overhead costs, which are not included in the comparative index, indicated that the cost of power in the recommended plan would be only 0.15 mill per kilowatt-hour greater. This increase was considered justified. Study 7B-7.512, with Passamaquoddy Bay in the lower pool, has the same comparative index as its opposite, study 7B-7.212, but slightly more energy. The recommended plan is preferred over study 7B-7.512 for the same reasons it is preferred over study 7B-7.212. In addition, the recommended plan serves public interest by making the larger pool and the one serving the greater number of communities (Passamaquoddy Bay) the upper pool.

f. Other Plans Considered.

(1) General. Comparative studies were made for about 100 separate arrangements. Sixty of these involved different sites for some of the project features. The remainder were similar to other studies except in the number of generating units or the number of filling and emptying gates. The studies included all reasonable sites for dams, about 20 sites for filling gates or emptying gates, and about 8 sites for a powerhouse. As these studies proceeded, it was possible by comparison to identify some factors and features which tended to produce a favorable comparative index for those arrangements in which they could be incorporated. Such features are discussed below.

(2) Arrangements Suggested in 1952. The studies included the principal features of the arrangements (Alternates 1 to 5) shown in the report "Details of Estimate of Cost-Comprehensive Investigation - Passamaquoddy Tidal Power Project - May 1952." Alternate 1 is the plan proposed by Dexter P. Cooper in 1928. Foundation exploration subsequent to the proposal of this plan showed that the location in Alternate 2 would be much more suitable with respect to the elevation of bedrock foundations for the powerhouse and the adjacent gates. Alternate 2, "Modified Cooper Plan" is similar to study 1-1.211 (plate 5-14) except in minor detail. It was found that the space between Treat Island and Dudley Island was too narrow for the total required capacity of emptying gates. Extending the width of these gates into the higher ground on the islands increased rock excavation which was in excess of requirements for fill material. It was found more economical to divide the 60 emptying gates with 20 of them between Treat and Dudley Islands and the remainder to be located on a rocky shoal at the opposite end of Treat Island. Consequently the location of the lock was shifted from Treat Island to Estes Head to remove it from the vicinity of the emptying gates. Alternates 3, 4, and 5 each involve a powerhouse headrace channel through Bar Harbor at the northwest side of Moose Island. This was found to require more excavation than would be necessary for the headrace in Carryingplace Cove unless Cobscook Bay were to be the upper pool and the powerhouse were to be limited to 15 units or less. Although location of the headrace in Bar Harbor would make possible a relatively economical dam across Western Passage at Kendall Head, neither abutment of the dam affords an economical site for a substantial number of gates.

(3) Head Harbour Passage Dams. Alternate 4 incorporates a dam across Head Harbour Passage from Deer Island to Pope Islet to Campobello Island. The location of this dam was modified to form the basis for the recommended plan. The part of the dam from Pope Islet to Deer Island was relocated to extend from Pope Islet to Indian Island and thence to Deer Island Point. This was found to be more economical because it permits the location of some of the filling gates on Deer Island Point, thereby reducing the significant power losses and larger excavation which would result if all the filling gates were located at Letite Passage. The remainder of the dam across Head Harbour Passage has been located at Green Islet instead of Pope Islet because less fill would be required and because the relatively shallow area between Pope Islet and Green Islet could be used for emptying gates. This site is relatively advantageous for gates for several reasons. The surface of bedrock is suitably high at the gate structure except for a short distance; Pope Islet and Green Islet constitute the gate abutments without the need for retaining walls to confine the fill in the adjacent dams; excavation for the channel would be moderate; and the edges of the excavation are not located on steep slopes where a cofferdam would be more costly. Several sites for dams abutting Indian Island were studied in various arrangements. Although a dam from the south end of Indian Island to Campobello Island would result in an arrangement costing about 5 percent less than the recommended plan, it was found that this saving would be more than offset by the decrease in power output due to the reduced pool area. However, in an arrangement dividing Passamaquoddy Bay between upper and lower pools (as shown on plate 5-9), it was found economical to use the dam between Indian Island and Campobello Island. A major reason for this difference lies in the relative areas of the pools. With Passamaquoddy Bay divided between the upper and lower pools, extension of the pool in Head Harbour Passage increases the size of the larger of the two pools. If Passamaquoddy Bay is not divided, as in the recommended plan, extension of the pool in Head Harbour Passage increases the size of the smaller pool with consequently greater benefit in the power output.

(4) Western Passage Dam. The dam in Western Passage at the tip of Deer Island Point was found to be relatively economical. It would require less fill than any dam across Western Passage except one at Kendall Head, and would be partially founded on rock and partially on stable sediments. In the recommended arrangement, its location would permit locating 50 of the 90 filling gates on Deer Island Point, discharging into Western Passage, thereby reducing the hydraulic losses which would occur if the pool in Passamaquoddy Bay were filled only through Letite Passage.

(5) Treatment in the Letite Passage Area. A number of possible locations for the dam and gates in the Letite Passage area were studied. The selected layout which has been incorporated in all the plans described herein is based on the combination of hydraulic efficiency and construction cost which would provide power at minimum cost. The gates would be located where the elevation of bedrock is suitable for structures and where the channels would generally expand both upstream and downstream of the gates and thus afford maximum hydraulic efficiency. The deep channel adjacent to McMaster Island is very narrow and thus makes possible a dam with a minimum volume of fill. Little Letite Passage is not suited for installation of gates because its small discharge capacity would justify only a few gates, which would be separated from the other gates.

(6) Powerhouse Sites. Powerhouse sites were considered in Carryingplace Cove, Bar Harbor, on Deer Island Point, Indian Island and Navy Island (St. Andrews Island). The use of most of these sites is dependent upon the general arrangement of the project and not suitable for separate discussion. In the recommended arrangement, however, the location of the powerhouse could be shifted from Carryingplace Cove to Deer Island Point by relocating the filling gates toward the base of the peninsula. The dam between Indian Island and Deer Island Point would also be relocated with the abutment between the powerhouse and the filling gates. Locating the powerhouse on Deer Island Point would substantially increase the volume of rock excavation and the cost of cofferdams. It was found that these items would hardly be offset by savings in haul distances and material handling even with an optimistic design for the slopes of the cofferdams. The site for a powerhouse and headrace in Bar Harbor has been briefly discussed in preceding subparagraph (2).

(7) Alternate Discharge to the Ocean. The economy of arranging the powerhouse to discharge alternately to the ocean and the lower pool was studied at sites in Carryingplace Cove (plate 5-14), Bar Harbor, on Deer Island Point and Indian Island. None of these arrangements had a comparative index as low as that of the recommended plan. When the powerhouse at Deer Island Point was studied with alternate discharge to the ocean, the tailrace gates were assumed to be located on Indian Island. The area between Deer Island Point and Indian Island would be a basin in the tailrace formed by dams at both sides with the water level controlled by the tailrace gates. These dams would serve as cofferdams for the basin side of the powerhouse and tailrace gates. Another plan, which was considered briefly after preparation

of the estimates shown on tables 5-2 and 5-3, is similar to the recommended plan except that both the powerhouse and the tailrace gates needed for alternate discharge to the ocean would be located on Deer Island Point. This plan would produce 5 percent more energy than the recommended plan neglecting hydraulic losses in the tailrace basin. Substantial losses appeared unavoidable, however, because the narrow width of Deer Island Point would make it necessary to place the tailrace gates close and parallel to the powerhouse. One set of tailrace gates would be opposite units at one end of the powerhouse, and the other set would be opposite the other end of the powerhouse making necessary two sharp turns in flow through the tailrace basin. The clear distance between the powerhouse and the tailrace gate structures would be as little as 700 feet at the south end and no more than 1,700 feet at the north end (compared with the estimated 2580-foot powerhouse length). This arrangement was not developed in detail and estimated because, on the basis of comparison with the recommended plan, it was evident that the additional costs for excavation and cofferdams plus the tailrace gates would not be justified by the increase in energy. This was especially evident as cofferdam studies were leading to the conclusion that the cofferdam design assumed at the beginning of the comparative studies would be inadequate, particularly for severe conditions like those at Deer Island Point.

(8) <u>Quoddy Roads</u>. The effect of including Quoddy Roads in the pool area was studied in a number of arrangements: study 4-6.33 (the recommended plan), study 6A-2.613, study 7B-7.22 and others. The studies involved comparing the cost of a dam in Quoddy Roads at Duck Point with the cost of a dam at Lubec Narrows, assuming the difference in the cost of the small locks to be negligible. The comparative costs for the dams were $2,200,000 for Quoddy Roads and $600,000 for Lubec Narrows. Although the dam at Quoddy Roads is more costly, it would result in the generation of more energy due to the increased area of the lower pool. It was computed that the additional energy in kilowatt-hours per year would be 31,000,000 in the recommended plan, and 10,000,000 in study 6A-2.613. This gave a comparative index for the additional energy of $0.052 and $0.160 respectively. On this basis, Quoddy Roads was included in the pool area for the recommended plan but was excluded from study 6A-2.613. In study 7B-7.22 the incremental cost of the Quoddy Roads site including an additional lock would be $2,300,000, the additional energy would be 39 million kw.-hr. per year and the incremental comparative index $0.059. A hole was drilled at a site for a lock at Lubec Narrows after the comparative studies were made. Drilling was stopped at el. -72 without reaching bedrock. At the recommended lock site in Quoddy Roads, bedrock is exposed at the surface and a drill hole indicated satisfactory subsurface conditions.

5-08 CONCLUSION

The studies described in this appendix indicated that:

(a) A single-pool project would not be suitable for the proposed international Passamaquoddy tidal power project because the cost of power would be higher than for a two-pool project, because generation would be intermittent, and because, for an equivalent amount of energy, peak generation rates would be considerably greater.

(b) Two single-pool projects with separate powerhouses could generate continuously but the cost of power would be higher than for a two-pool plan.

(c) Two pools operated as a double single-pool project with a common powerhouse would not increase energy output sufficiently compared with a simple two-pool plan to offset the lack of dependable capacity and the additional cost.

(d) Use of alternate discharge to the ocean with a two-pool plan would increase the cost of tidal power, and consequently was not adopted for the purposes of this survey.

(e) Study 4-6.33, the recommended plan, is the best of the two-pool plans studied. This plan was approved for further study by both the United States and Canadian Sections of the International Joint Commission by letters dated 27 January 1958.

APPENDIX 5

TABLES

TABLE 5-1

UNIT PRICES USED FOR COMPARATIVE COSTS

DESCRIPTION	UNIT	UNIT PRICES
EXCAVATION BY SHOVELS AND TRUCKS (BANK MEASURE)*		
(Including cofferdam pumping and 3500-ft. delivery)		
Earth Excavation	cu. yd.	$ 0.65
Rock Excavation	cu. yd.	1.48
FILL FROM BORROW BY SHOVELS AND TRUCKS (LOOSE MEASURE)*		
(Including 3500-ft. delivery)		
Earthfill	cu. yd.	0.63
Rockfill	cu. yd.	.97
TRANSPORTATION OF ABOVE (LOOSE MEASURE)*		
By truck (earth or rock)	yd-mile	0.15
By dump scow (rock) including loading, towing and dumping		
To 1 mile	cu. yd.	0.294
1 to 3 miles	cu. yd.	0.381
4 to 8 miles	cu. yd.	0.553
9 to 13 miles	cu. yd.	0.726

DERRICK STONE AND RIPRAP (LOOSE MEASURE)*

(Including production, transportation, and placement)

Distance	Unit	By Truck	By Scows and Lighters
2 miles	cu. yd.	$ 1.58	$2.20
6 miles	cu. yd.	2.20	2.40
10 miles	cu. yd.	2.82	2.59
14 miles	cu. yd.	3.44	2.79

DESCRIPTION	UNIT	UNIT PRICES
FOUNDATION CONCRETE (needed where surface of bedrock is low for structures)	cu. yd.	$16.00

* In earth, bank measure = loose measure
In rock, bank measure x 1.5 = loose measure

TABLE 5-1 (Continued)

DESCRIPTION	UNIT	UNIT PRICES
GATES, (30' x 30' VENTURI TYPE) INCLUDING STRUCTURES AND EQUIPMENT		
Single Filling Type	per gate	$ 148,300
Single Emptying Type	per gate	162,000
Double Filling Type	per pair	247,400
Double Emptying Type	per pair	253,900
Concrete in Gate Abutments:		
Gravity Type	cu. yd.	26.00
Semi-gravity Type (Incl. reinforcement)	cu. yd.	38.00
POWERHOUSE AND EQUIPMENT		
Generator and Exciter	per unit	$1,000,000
Fixed-blade Turbine and Governor	per unit	1,250,000
Cathodic Protection	per unit	200,000
Remaining Mech.-Elec. Equipment	per unit	630,000
Substructure - (Unit Bays)	per unit	932,000
Superstructure - (Unit Bays)	per unit	218,000
Miscellaneous	per unit	70,000
TOTAL - POWERHOUSE AND EQUIPMENT	per unit	$4,300,000
SERVICE BAY STRUCTURES	each	600,000
LOCKS - STRUCTURE AND EQUIPMENT (Locks for moderate increase in size of vessels)		
415 x 60 x 21 ft. (gravity wall type)		$6,105,000
370 x 50 x 19 ft. (lined rock walls)		2,182,000
95 x 25 x 10 ft. (gravity wall type)		942,000

Some typical unit prices from which the above costs of structures were obtained are shown below:

Concrete, (not incl. reinf.)		
In Gate Structures	per cu. yd.	$ 32.50
In Powerhouse Substructure	per cu. yd.	32.10
In Locks (gravity wall type)	per cu. yd.	24.60
Steel Reinforcement	per lb.	0.15
Structural Steel	per lb.	0.30
Gates (incl. wheels, bearings, etc.)	per lb.	0.50

Note: All prices in comparative estimates are estimated contract prices, i.e., without contingencies or Government costs.

TABLE 5-2

INITIAL COMPARATIVE STUDIES
SUMMARY OF COSTS AND ENERGY

Study	4-6.22	4-6.52	6A-2.613	7B-7.22	1-1.211*
Plate	5-10	5-11	5-12	5-13	5-14
Passamaquoddy Bay Level	High	Low	Divided	High	High
Powerhouse location	Carry-ingplace	Carry-ingplace	St. Andrews	Carry-ingplace	Carry-ingplace
Number of turbines	30	30	30	20	30
Number, filling gates	110	70	100	120	120
Number, emptying gates	70	110	80	60	60
Pool areas, square miles					
Upper	100.5	50.7	87.2	100.5	99.0
Lower	39.4	95.2	51.6	28.9	28.9
Total	139.9	145.9	138.8	129.4	127.9
Ratio, smaller pool area to larger	0.392	0.533	0.592	0.287	0.292
Square miles per unit	4.66	4.86	4.63	6.47	4.26
Million kw.-hr./yr./unit	61.8	65.7	65.0	69.0	61.5
Million kw.-hr./yr.					
Gross	1,850	1,970	1,950	1,380	1,850
Reduced for losses	1,850	1,970	1,880	1,380	1,800
Fill (millions cu. yds.)					
Dams	41.0	39.6	50.1	22.9	25.3
Cofferdams	18.8	18.4	14.0	13.7	7.0
Costs in $1,000,000					
Excavation & fill	81.9	87.3	96.4	55.3	73.9
Locks	14.1	14.1	6.3	9.2	9.3
Gates	26.8	26.9	22.9	24.8	37.9
Powerhouse	130.2	130.2	130.2	87.2	130.2
Relocations	1.0	1.0	0.0	1.0	1.0
Total comparative cost	254.0	259.5	255.8	177.5	252.3
Comparative index (in dollars per annual kw.-hr.)	$0.137	$0.132	$0.136	$0.129	$0.140

* In this plan, the tailrace would discharge alternately to the ocean and the lower pool.

TABLE 5-3

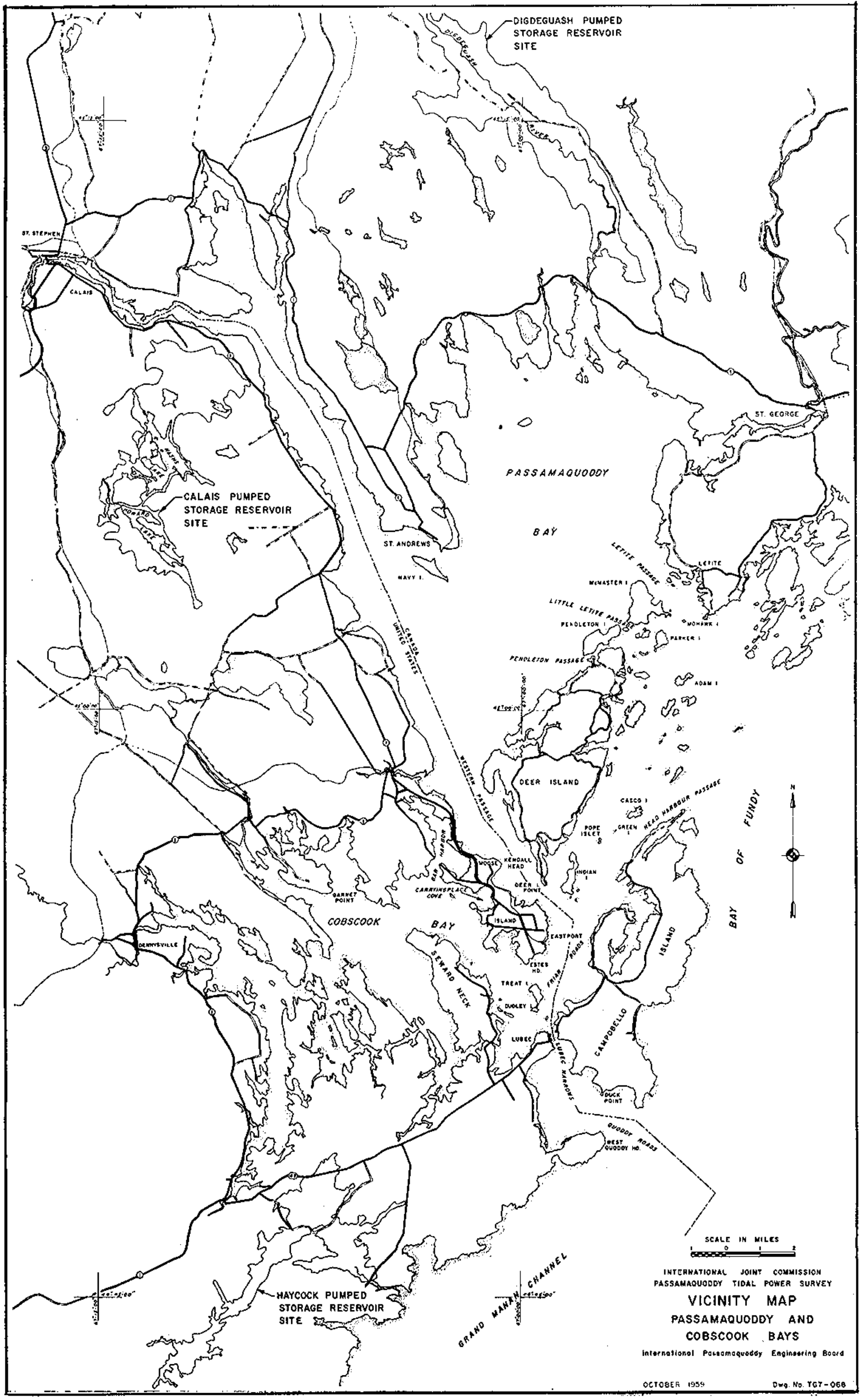
FINAL COMPARATIVE STUDIES
SUMMARY OF COSTS AND ENERGY

Study	4-6.33	7B-7.212	7B-7.512
Plate	5-7	5-8	5-9
Passamaquoddy Bay level	High	High	Low
Powerhouse location	Carry-ingplace	Carry-ingplace	Carry-ingplace
Number of turbines	30	20	20
Number, filling gates	90	80	50
Number, emptying gates	70	50	80
Pool areas, square miles			
Upper	100.5	100.5	42.0
Lower	41.0	31.9	94.5
Total	141.5	132.4	136.5
Average Annual Energy in billions of kw.-hr.	1.774	1.320	1.379
Costs in $1,000,000			
Excavation & fill	76.9	52.0	58.7
Locks	10.2	10.2	10.2
Gates	24.7	19.8	20.4
Powerhouse	130.2	87.2	87.2
Relocations	1.0	1.0	1.0
Total comparative cost	243.0	170.2	177.5
Comparative index (in dollars per annual kw.-hr.)	$0.137	$0.129	$0.129

Note: An appraisal of operating and overhead costs, which are not included in the comparative indexes above, indicated that the cost of power in study 4-6.33 (the recommended plan) would be 0.15 mill per kilowatt-hour greater than for study 7B-7.212 due to greater output.

APPENDIX 5

PLATES


DIGDEGUASH PUMPED STORAGE RESERVOIR SITE
ST. STEPHEN
CALAIS
CALAIS PUMPED STORAGE RESERVOIR SITE
ST. GEORGE
PASSAMAQUODDY
BAY
ST. ANDREWS
NAVY I.
LETITE PASSAGE
LETITE
McMASTER I.
LITTLE LETITE PASSAGE
PENDLETON I.
MOHAWK I.
PARKER I.
PENDLETON PASSAGE
ADAM I.
CANADA
UNITED STATES
WESTERN PASSAGE
DEER ISLAND
CASCO I.
HEAD HARBOUR PASSAGE
POPE ISLET
GREEN I.
BAY OF FUNDY
KENDALL HEAD
DEER POINT
INDIAN I.
GARNET POINT
CARRYINGPLACE COVE
COBSCOOK
BAY
ISLAND
EASTPORT
DENNYSVILLE
SEWARD NECK
ESTES HD.
TREAT I.
FRIAR ROADS
DUDLEY I.
LUBEC
LUBEC NARROWS
CAMPOBELLO
ISLAND
DUCK POINT
QUODDY ROADS
WEST QUODDY HD.
GRAND MANAN CHANNEL
HAYCOCK PUMPED STORAGE RESERVOIR SITE
SCALE IN MILES
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
VICINITY MAP
PASSAMAQUODDY AND
COBSCOOK BAYS
International Passamaquoddy Engineering Board
OCTOBER 1959
Dwg. No. TG7-066

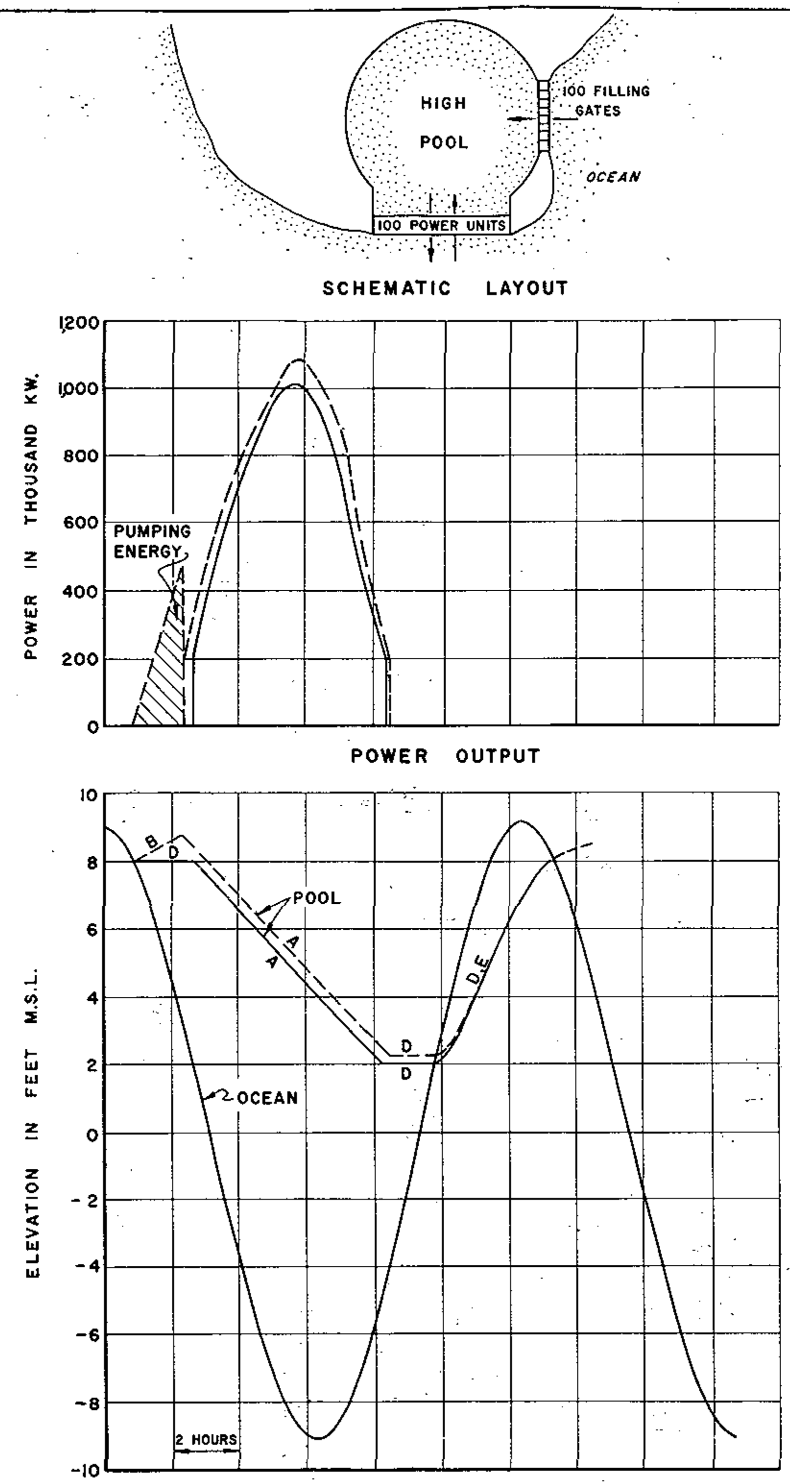


FIG. 1 - SINGLE HIGH POOL
---- WITH PUMPING
—— WITHOUT PUMPING

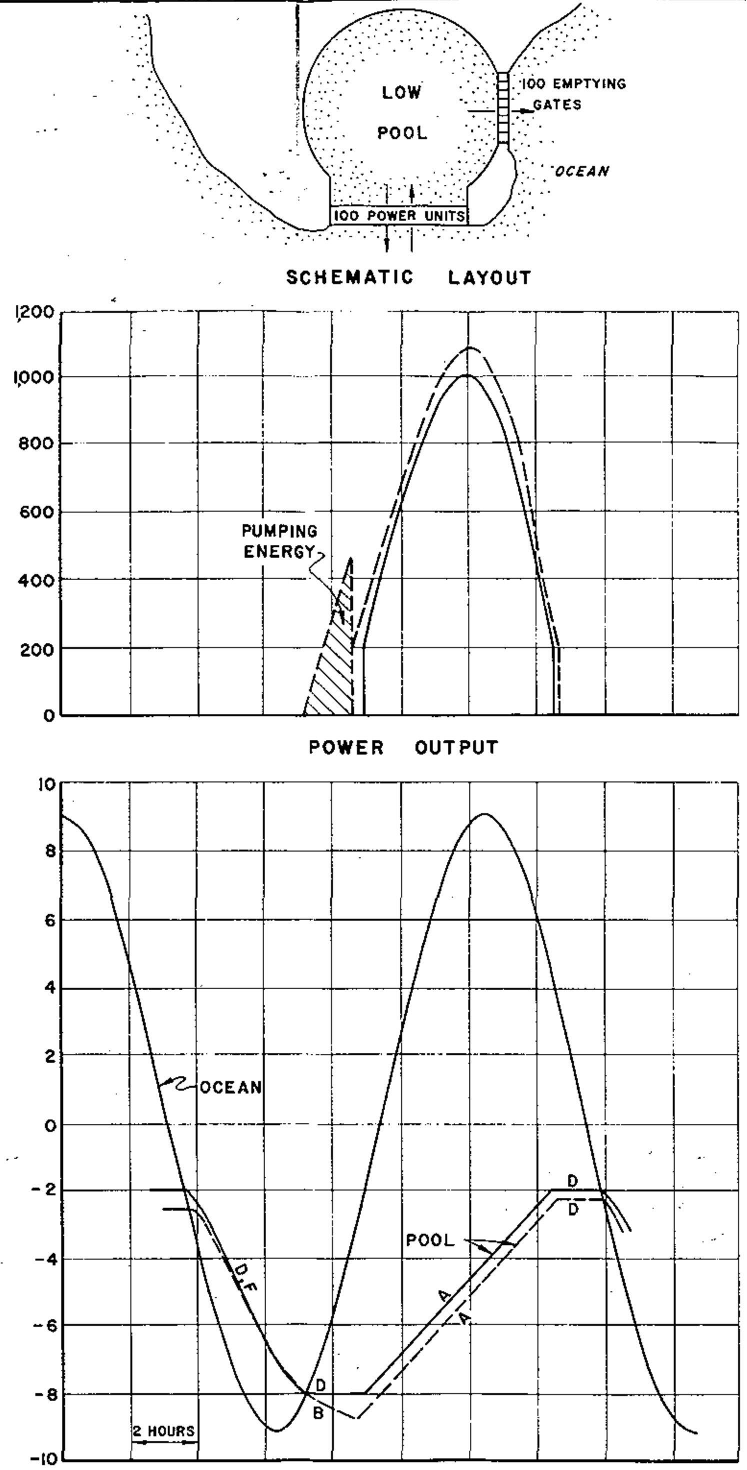


FIG. 2 - SINGLE LOW POOL
---- WITH PUMPING
—— WITHOUT PUMPING

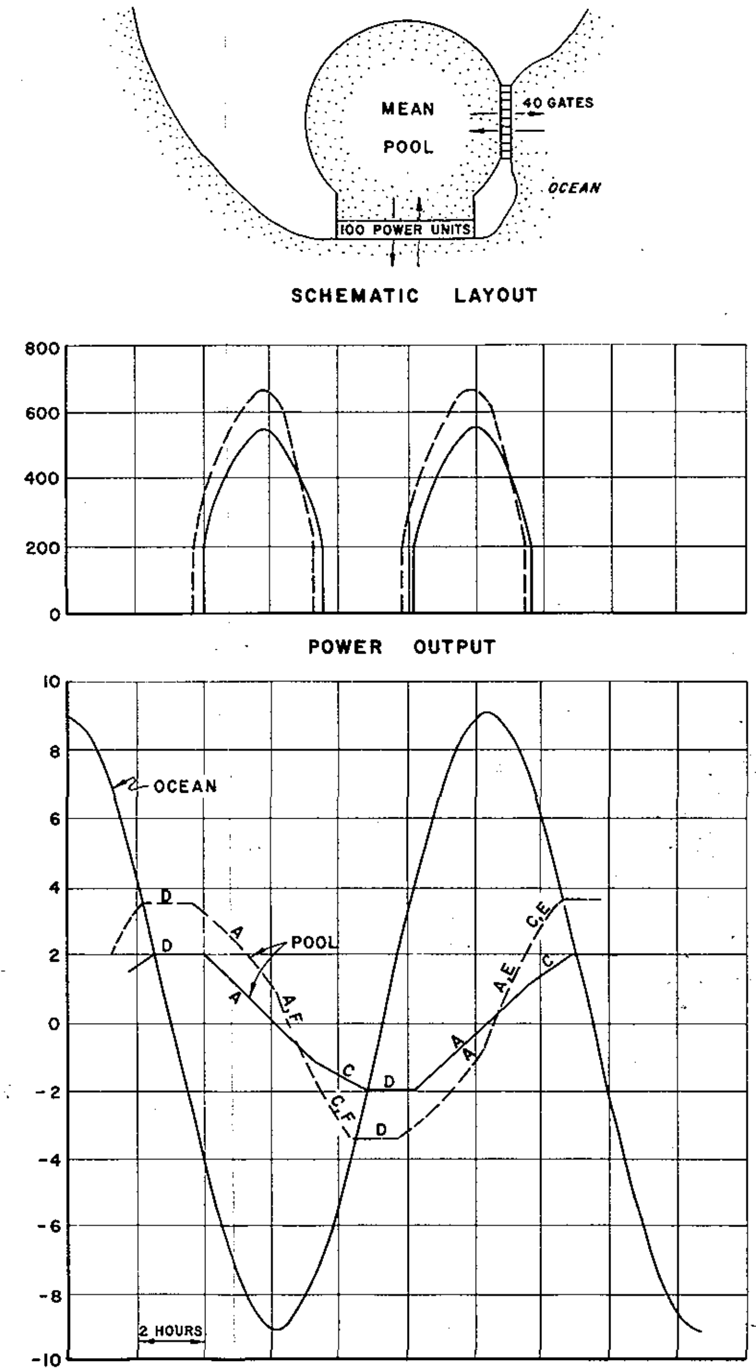


FIG. 3 - SINGLE MEAN POOL
---- WITH GATES
—— WITHOUT GATES

NOTES	
A	POWER UNITS AS TURBINES
B	POWER UNITS AS PUMPS
C	POWER UNITS AS ORIFICES
D	POWER UNITS INOPERATIVE
E	FILLING GATES OPEN
F	EMPTYING GATES OPEN

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

TIDAL POWER PROJECT

METHODS OF OPERATION
SHEET 1

International Passamaquoddy Engineering Board

OCTOBER 1959 Dwg. No. T67-069

PLATE 5-2

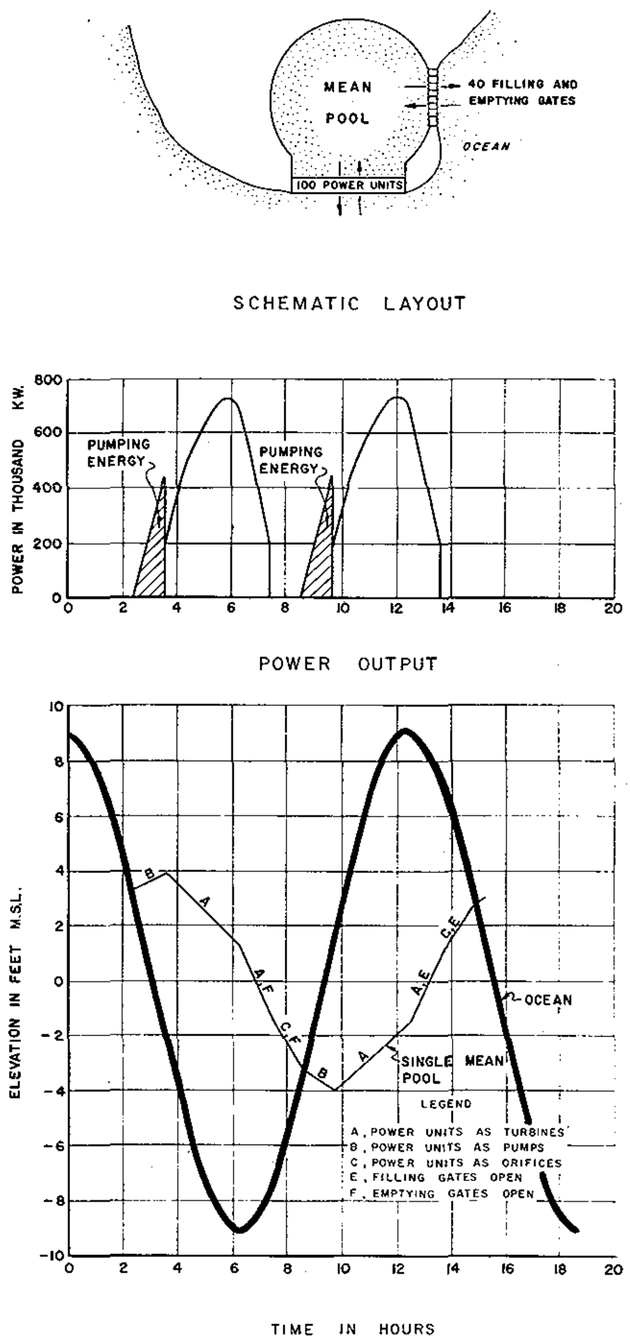


FIG. 1 - SINGLE MEAN POOL WITH GATES & PUMPING

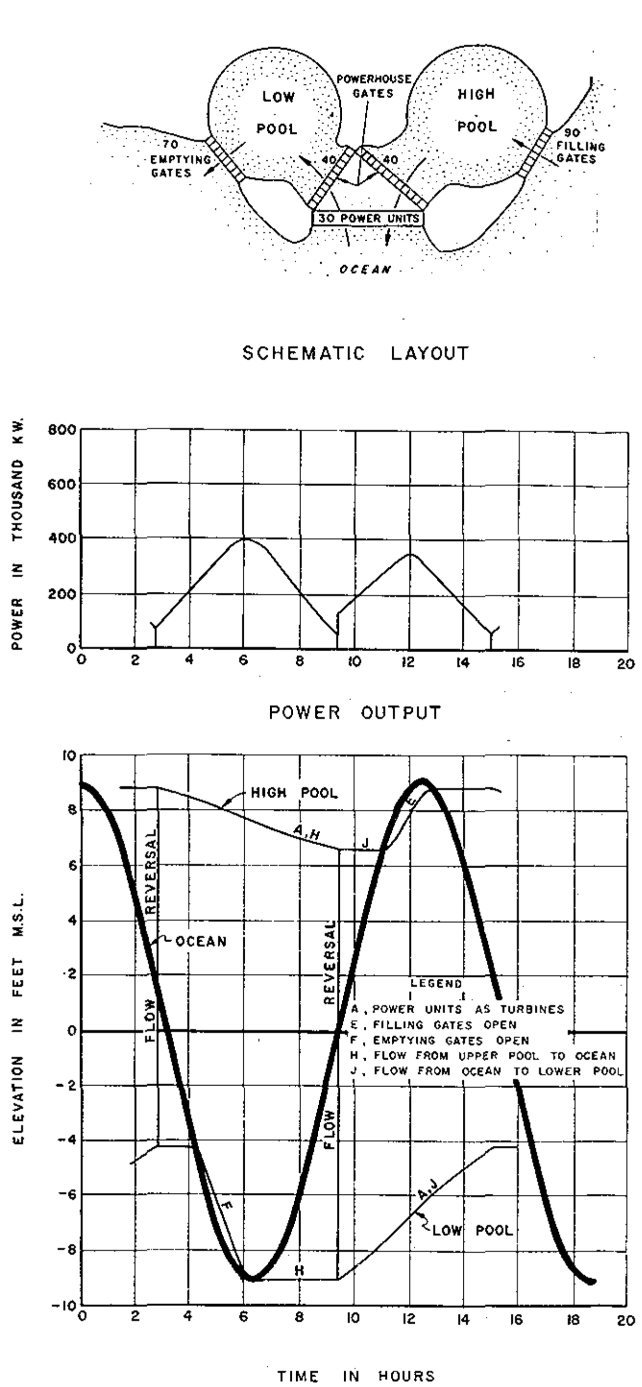


FIG. 2 - DOUBLE SINGLE POOL WITH A COMMON POWERHOUSE

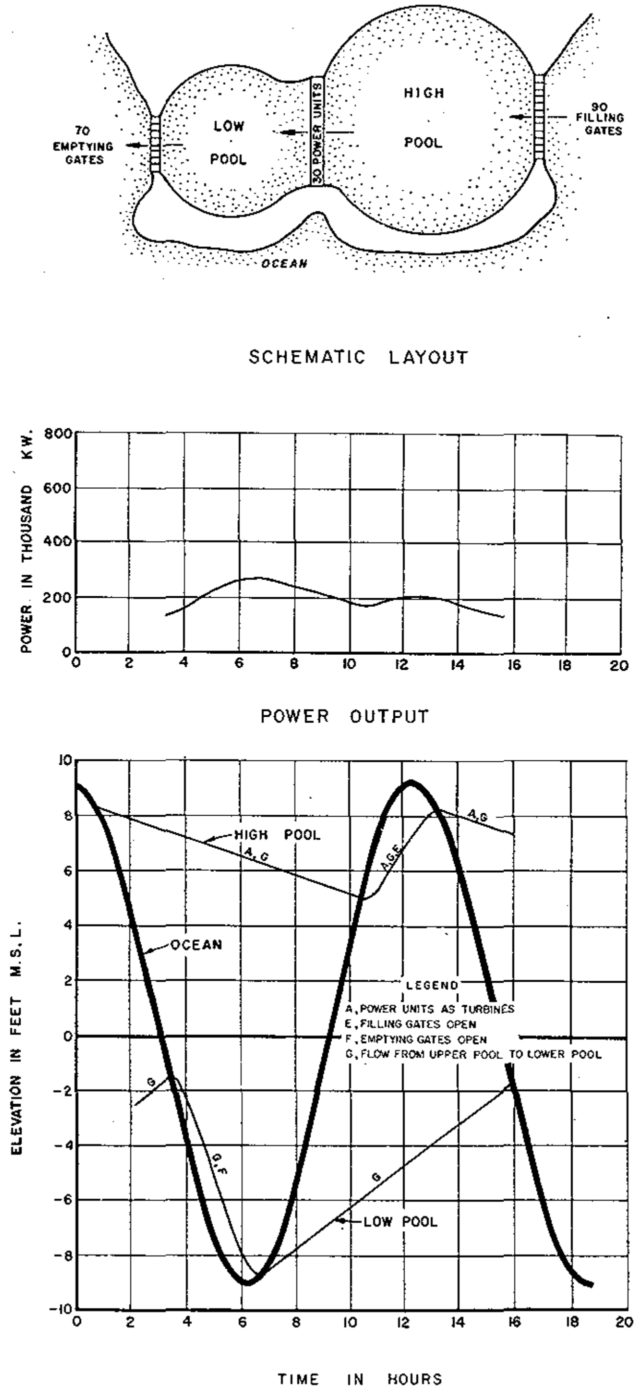


FIG. 3 - NORMAL TWO POOL

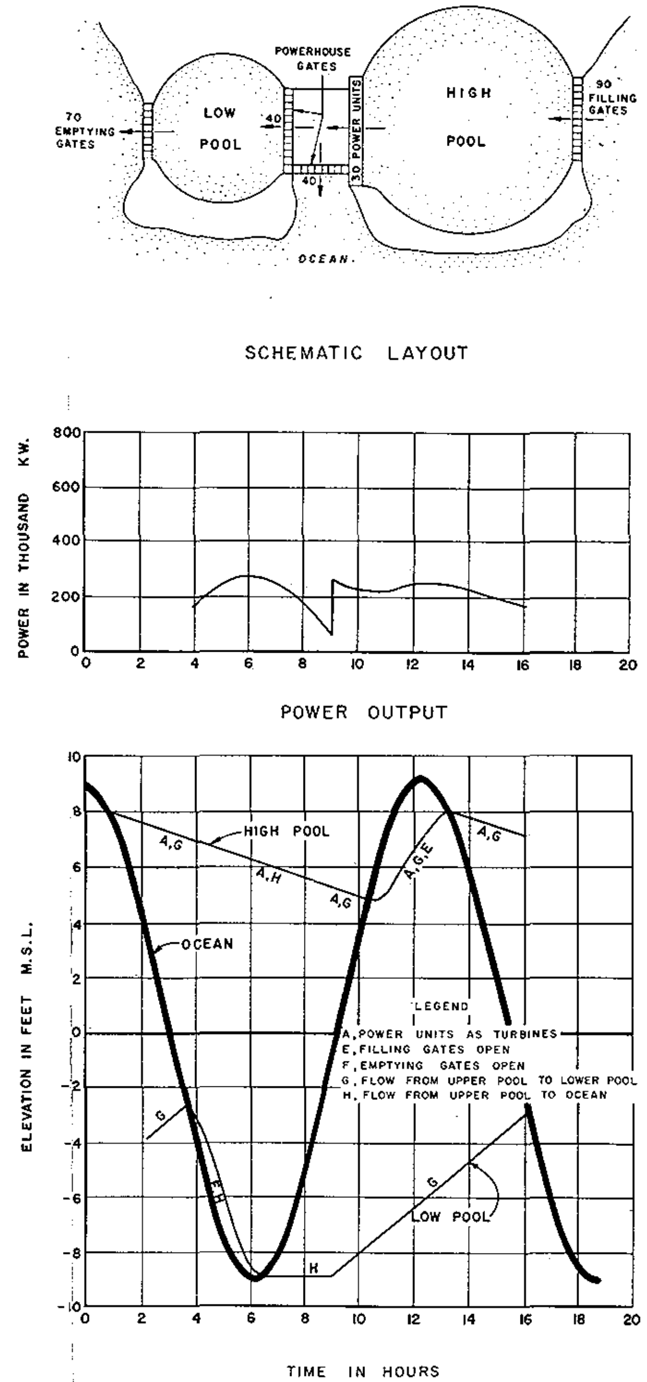


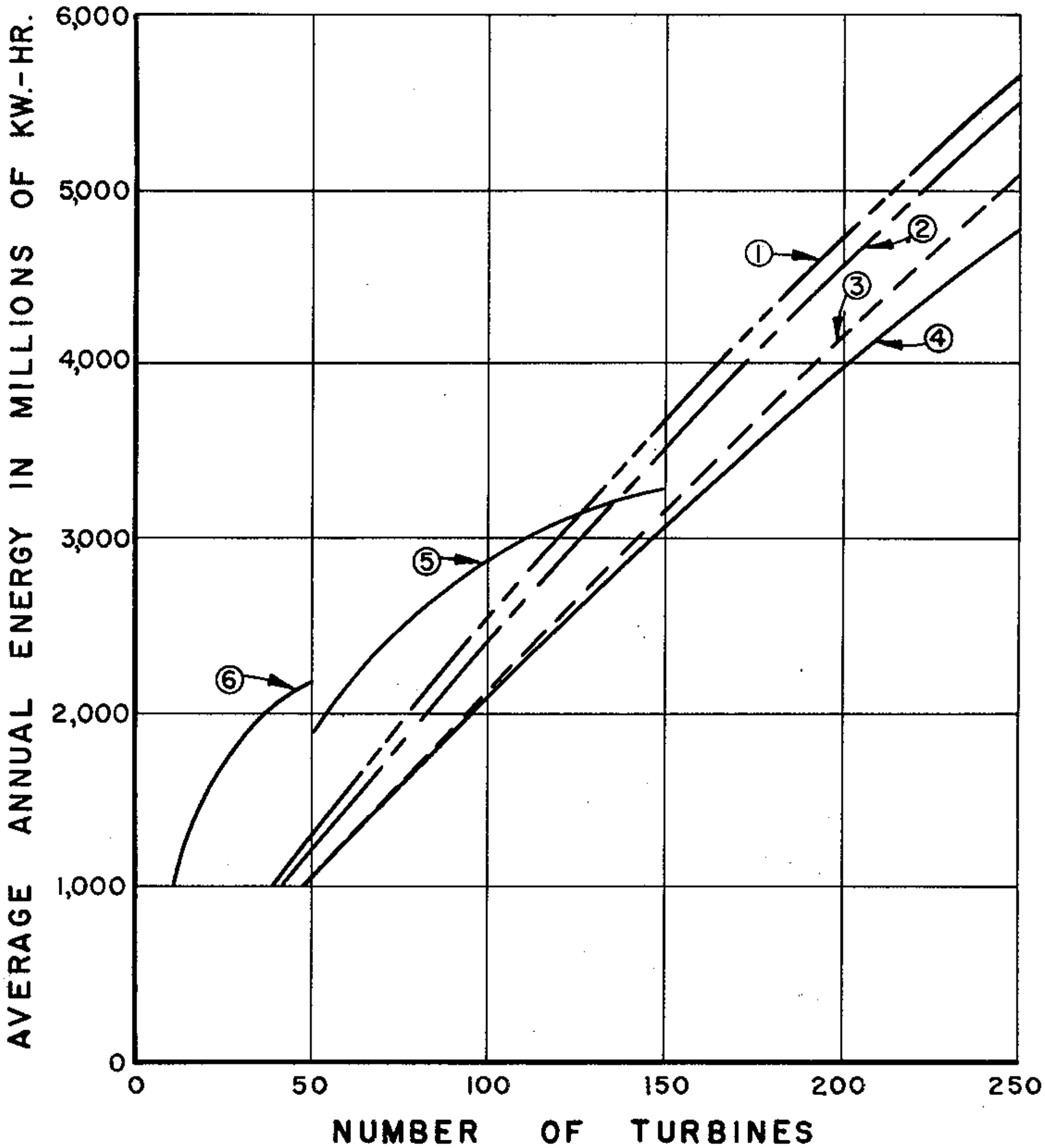
FIG. 4 - TWO POOL WITH DISCHARGE TO OCEAN

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

TIDAL POWER PROJECT
METHODS OF OPERATION
SHEET 2

International Passamaquoddy Engineering Board

OCTOBER 1959 Dwg. No. TG7-070



NOTES:

ENERGY COMPUTED FOR POOL AREA OF 146 sq. mi.

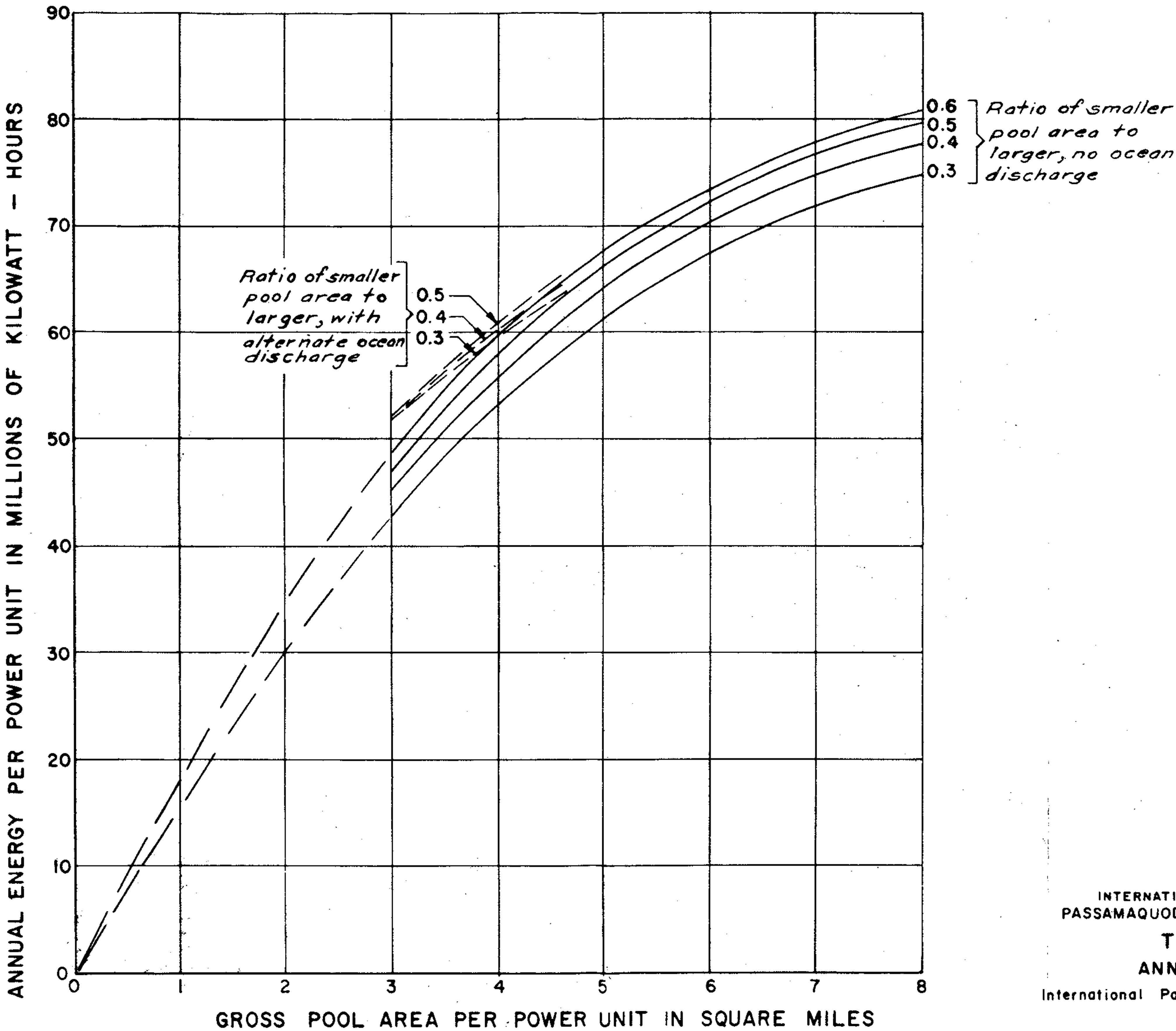
① SINGLE MEAN POOL, 40 GATES WITH PUMPING
② " " " 40 GATES WITHOUT PUMPING
③ " " " NO GATES WITH PUMPING
④ " " " NO GATES WITHOUT PUMPING
⑤ SINGLE HIGH POOL 100 GATES WITHOUT PUMPING
⑥ TWO POOL PLAN

INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

SINGLE - POOL
ANNUAL ENERGY

International Passamaquoddy Engineering Board

OCTOBER 1959 Dwg. No. TG7-071



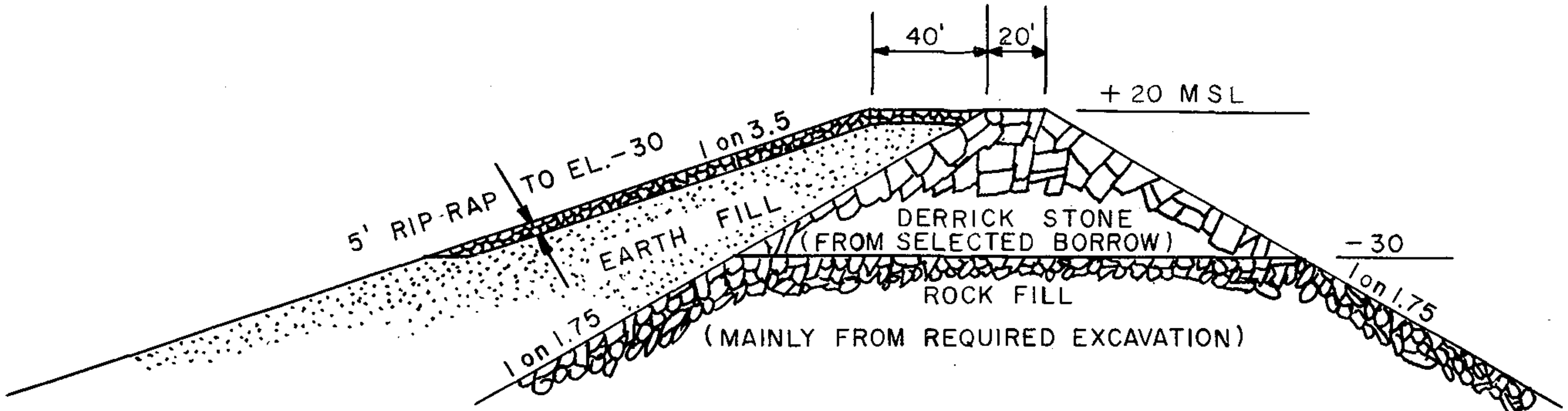
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

TWO - POOL

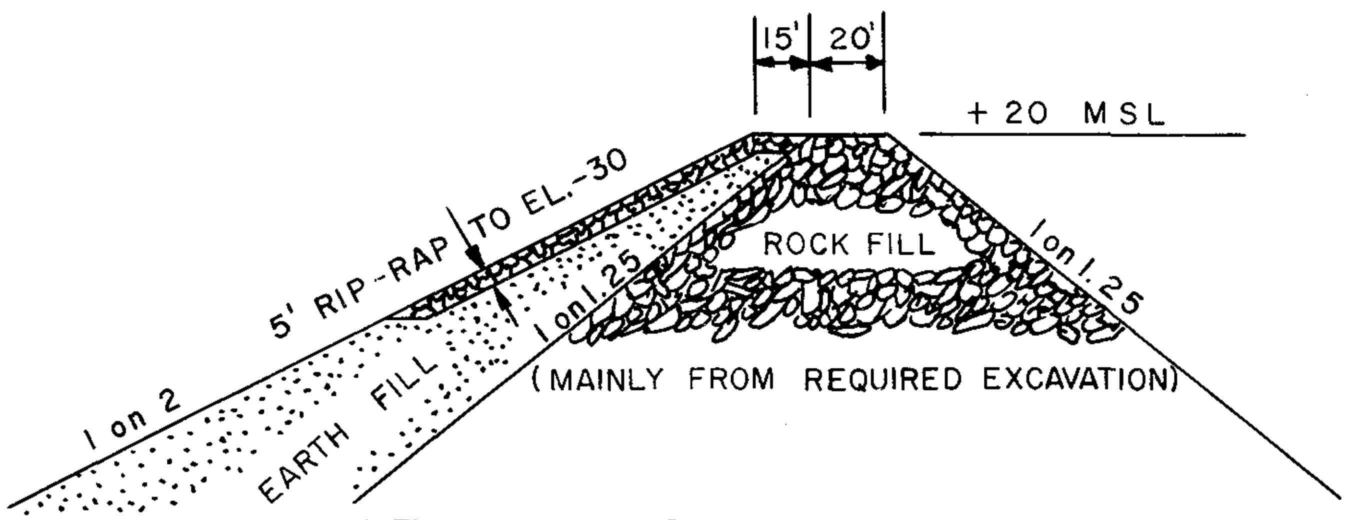
ANNUAL ENERGY

International Passamaquoddy Engineering Board

OCTOBER 1959 Dwg. No. TG7-072



TENTATIVE CROSS SECTION OF PERMANENT DAMS



TENTATIVE CROSS SECTION OF COFFERDAMS

NOTE:
These cross sections used only for comparative estimates of various sites.

SCALE 1" = 50'

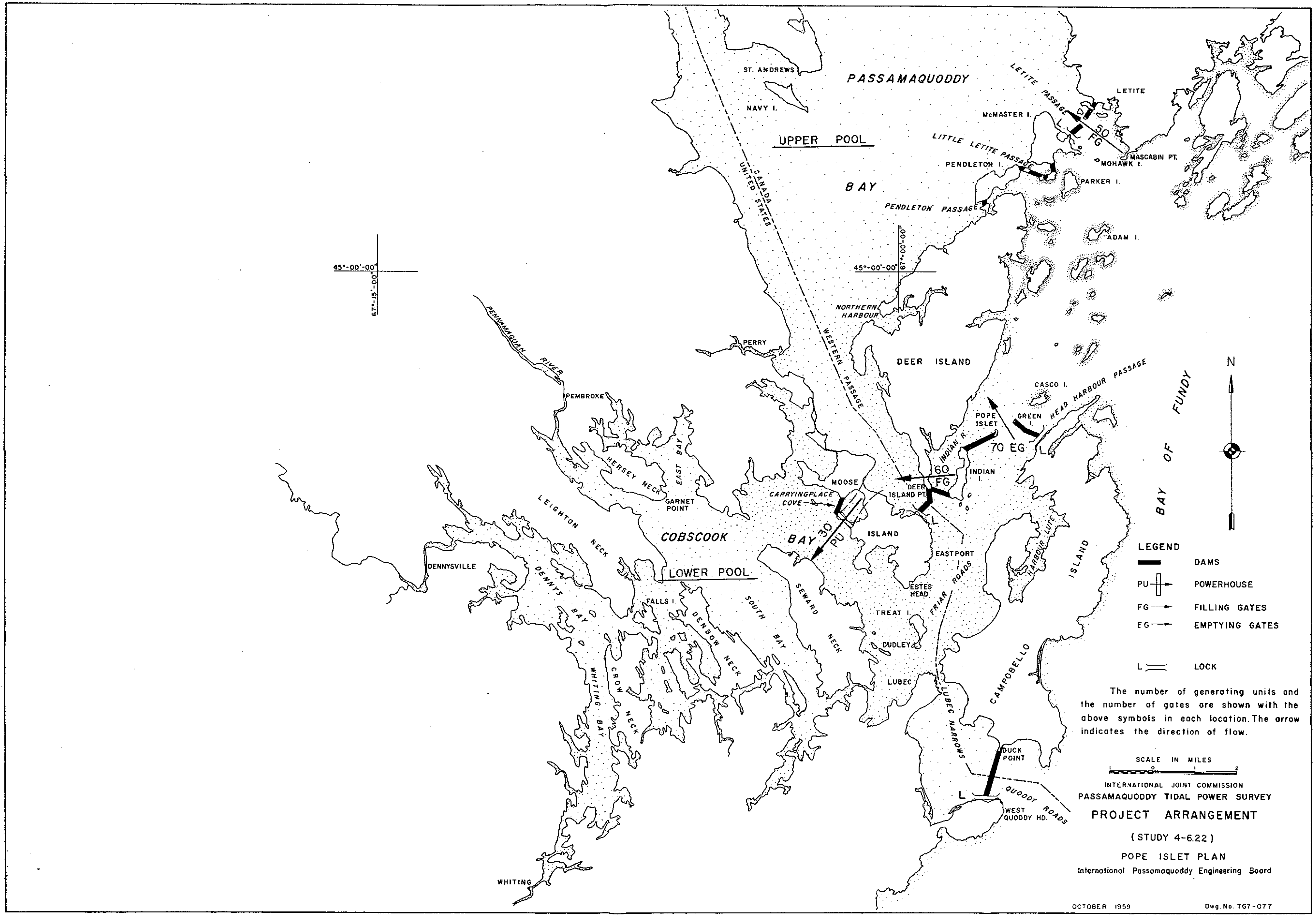
25 0 50 100 FT.

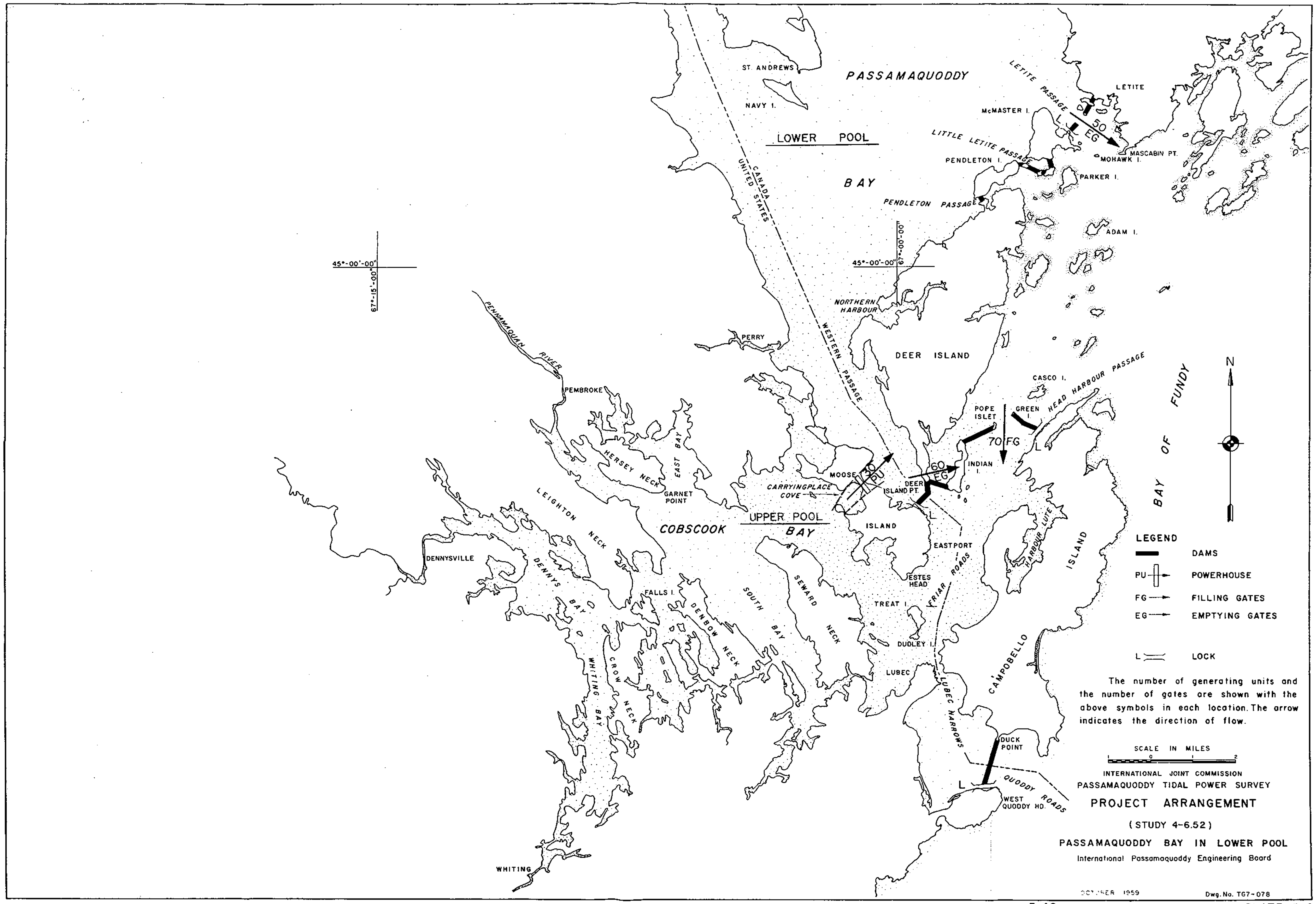
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

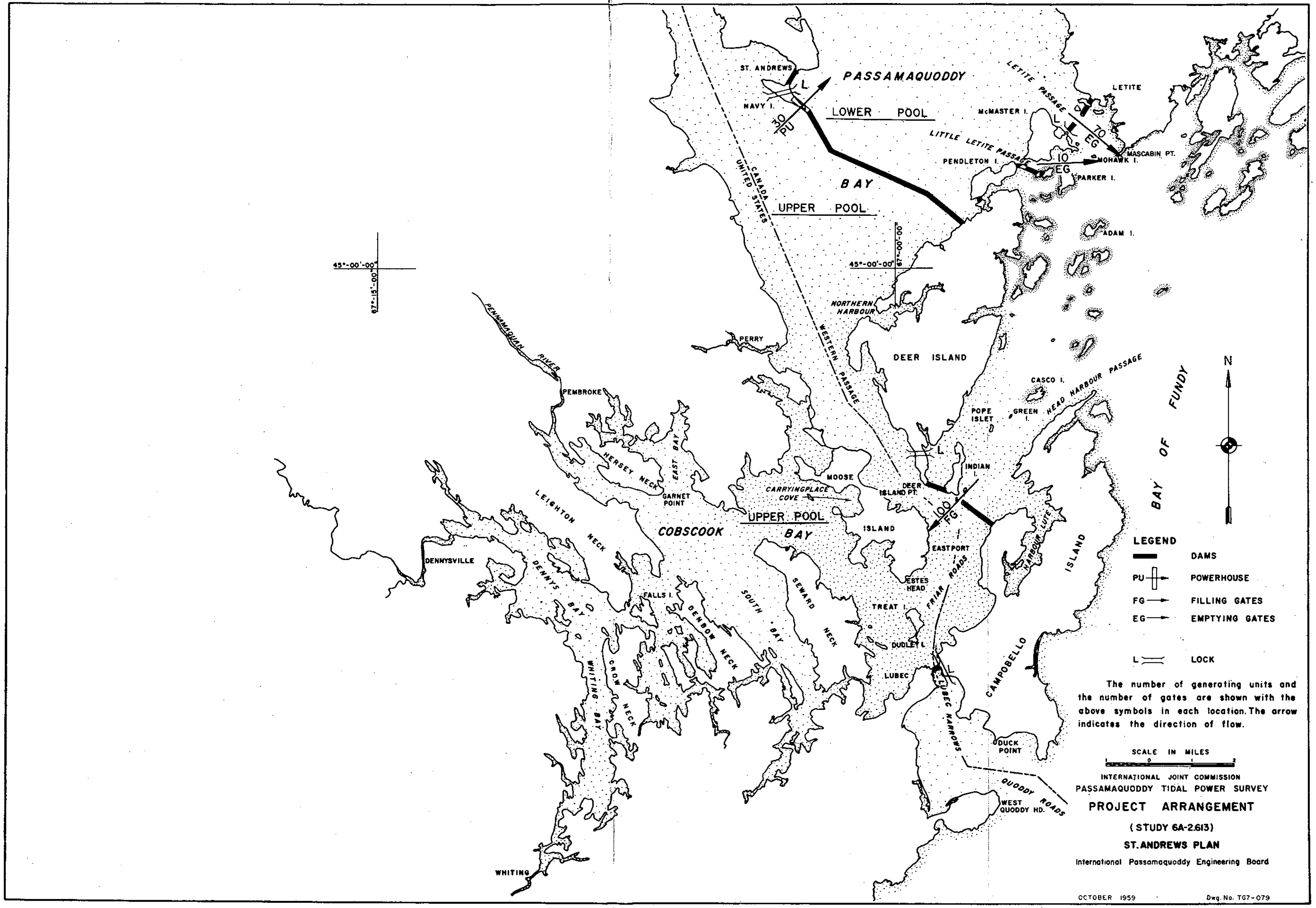
CROSS SECTION OF DAMS FOR COMPARATIVE STUDIES

INT'L. PASSAMAQUODDY ENGR. BOARD

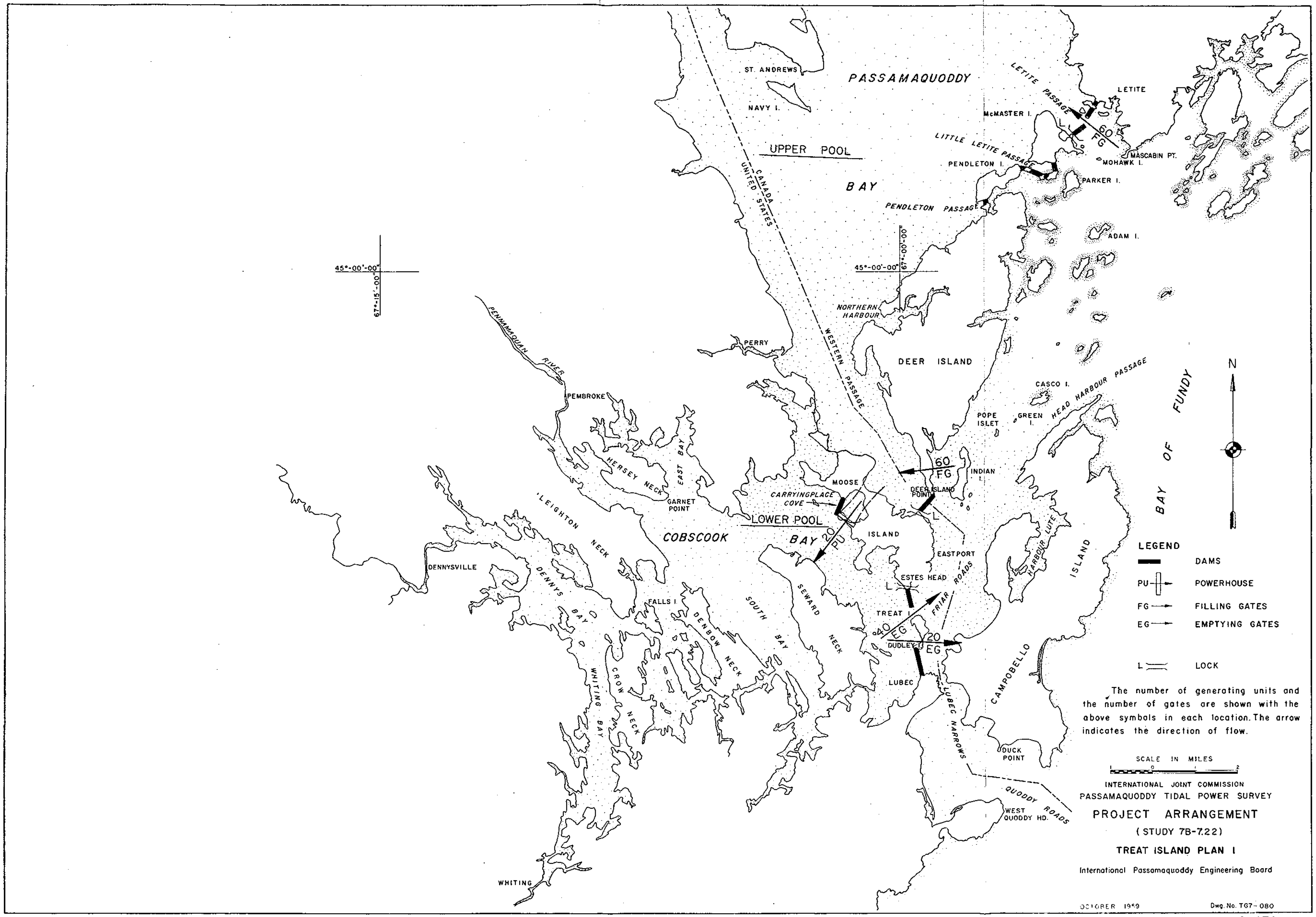
OCTOBER 1959 Dwg. No. TG7-073


PASSAMAQUODDY
UPPER POOL
BAY
ST. ANDREWS
NAVY I.
LETITE PASSAGE
LETITE
McMASTER I.
50 FG
LITTLE LETITE PASSAGE
PENDLETON I.
MASCABIN PT.
MOHAWK I.
PARKER I.
PENDLETON PASSAGE
ADAM I.
CANADA
UNITED STATES
45°-00'-00"
67°-00'-00"
67°-15'-00"
NORTHERN HARBOUR
WESTERN PASSAGE
PERRY
DEER ISLAND
PENNAMAQUAN RIVER
PEMBROKE
CASCO I.
HEAD HARBOUR PASSAGE
POPE ISLET
GREEN I.
70 EG
INDIAN R.
INDIAN I.
60 FG
DEER ISLAND PT.
MOOSE
CARRYINGPLACE COVE
HERSEY NECK
EAST BAY
GARNET POINT
LEIGHTON NECK
COBSCOOK
BAY
30 PU
ISLAND
EASTPORT
LOWER POOL
DENNYSVILLE
DENNYS BAY
FALLS I.
DENBOW NECK
SOUTH BAY
SEWARD NECK
ESTES HEAD
FRIAR ROADS
HARBOUR LUTE
ISLAND
BAY OF FUNDY
TREAT I.
DUDLEY I.
LUBEC
CROW NECK
WHITING BAY
CAMPOBELLO
LUBEC NARROWS
DUCK POINT
QUODDY ROADS
WEST QUODDY HD.
WHITING
N
LEGEND
DAMS
PU POWERHOUSE
FG FILLING GATES
EG EMPTYING GATES
L LOCK
The number of generating units and the number of gates are shown with the above symbols in each location. The arrow indicates the direction of flow.
SCALE IN MILES
1 0 1 2
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
PROJECT ARRANGEMENT
(STUDY 4-6.22)
POPE ISLET PLAN
International Passamaquoddy Engineering Board
OCTOBER 1959
Dwg. No. TG7-077


PASSAMAQUODDY
BAY
LOWER POOL
ST. ANDREWS
NAVY I.
LETITE PASSAGE
LETITE
McMASTER I.
50 EG
L
LITTLE LETITE PASSAGE
PENDLETON I.
MOHAWK I.
MASCABIN PT.
PARKER I.
PENDLETON PASSAGE
ADAM I.
CANADA
UNITED STATES
45°-00'-00"
67°-00'-00"
67°-15'-00"
NORTHERN HARBOUR
WESTERN PASSAGE
PERRY
DEER ISLAND
PENNAMAQUAN RIVER
PEMBROKE
CASCO I.
HEAD HARBOUR PASSAGE
POPE ISLET
GREEN I.
70 FG
L
BAY OF FUNDY
N
HERSEY NECK
EAST BAY
GARNET POINT
MOOSE
30 PU
60 EG
INDIAN I.
DEER ISLAND PT.
CARRYINGPLACE COVE
LEIGHTON NECK
UPPER POOL
COBSCOOK BAY
ISLAND
L
EASTPORT
DENNYSVILLE
DENNYS BAY
FALLS I.
DENBOW NECK
SOUTH BAY
SEWARD NECK
ESTES HEAD
FRIAR ROADS
HARBOUR LUTE
ISLAND
TREAT I.
DUDLEY I.
LUBEC
CAMPOBELLO
WHITING BAY
CROW NECK
LUBEC NARROWS
DUCK POINT
L
QUODDY ROADS
WEST QUODDY HD.
WHITING
LEGEND
DAMS
PU POWERHOUSE
FG FILLING GATES
EG EMPTYING GATES
L LOCK
The number of generating units and the number of gates are shown with the above symbols in each location. The arrow indicates the direction of flow.
SCALE IN MILES
1 0 1 2
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
PROJECT ARRANGEMENT
(STUDY 4-6.52)
PASSAMAQUODDY BAY IN LOWER POOL
International Passamaquoddy Engineering Board
OCTOBER 1959
Dwg. No. TG7-078



PASSAMAQUODDY BAY
LOWER POOL
UPPER POOL
ST. ANDREWS
NAVY I.
30 PU
L
LETITE PASSAGE
LETITE
McMASTER I.
LITTLE LETITE PASSAGE
PENDLETON I.
70 EG
10 EG
MASCABIN PT.
MOHAWK I.
PARKER I.
ADAM I.
CANADA
UNITED STATES
45°-00'-00"
67°-15'-00"
67°-00'-00"
NORTHERN HARBOUR
WESTERN PASSAGE
DEER ISLAND
PERRY
PENNAMAQUAN RIVER
PEMBROKE
CASCO I.
HEAD HARBOUR PASSAGE
POPE ISLET
GREEN I.
BAY OF FUNDY
N
HERSEY NECK
EAST BAY
GARNET POINT
MOOSE
CARRYINGPLACE COVE
UPPER POOL
COBSCOOK BAY
ISLAND
DEER ISLAND PT.
INDIAN I.
100 FG
EASTPORT
LEIGHTON NECK
DENNYSVILLE
DENNYS BAY
FALLS I.
DENBOW NECK
SOUTH BAY
SEWARD NECK
ESTES HEAD
TREAT I.
FRIAR ROADS
HARBOUR LUTE
ISLAND
DUDLEY I.
LUBEC
CAMPOBELLO
WHITING BAY
CROW NECK
LUBEC NARROWS
DUCK POINT
QUODDY ROADS
WEST QUODDY HD.
WHITING
LEGEND
DAMS
PU POWERHOUSE
FG FILLING GATES
EG EMPTYING GATES
L LOCK
The number of generating units and the number of gates are shown with the above symbols in each location. The arrow indicates the direction of flow.
SCALE IN MILES
1 0 1 2
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
PROJECT ARRANGEMENT
(STUDY 6A-2.613)
ST. ANDREWS PLAN
International Passamaquoddy Engineering Board
OCTOBER 1959
Dwg. No. TG7-079



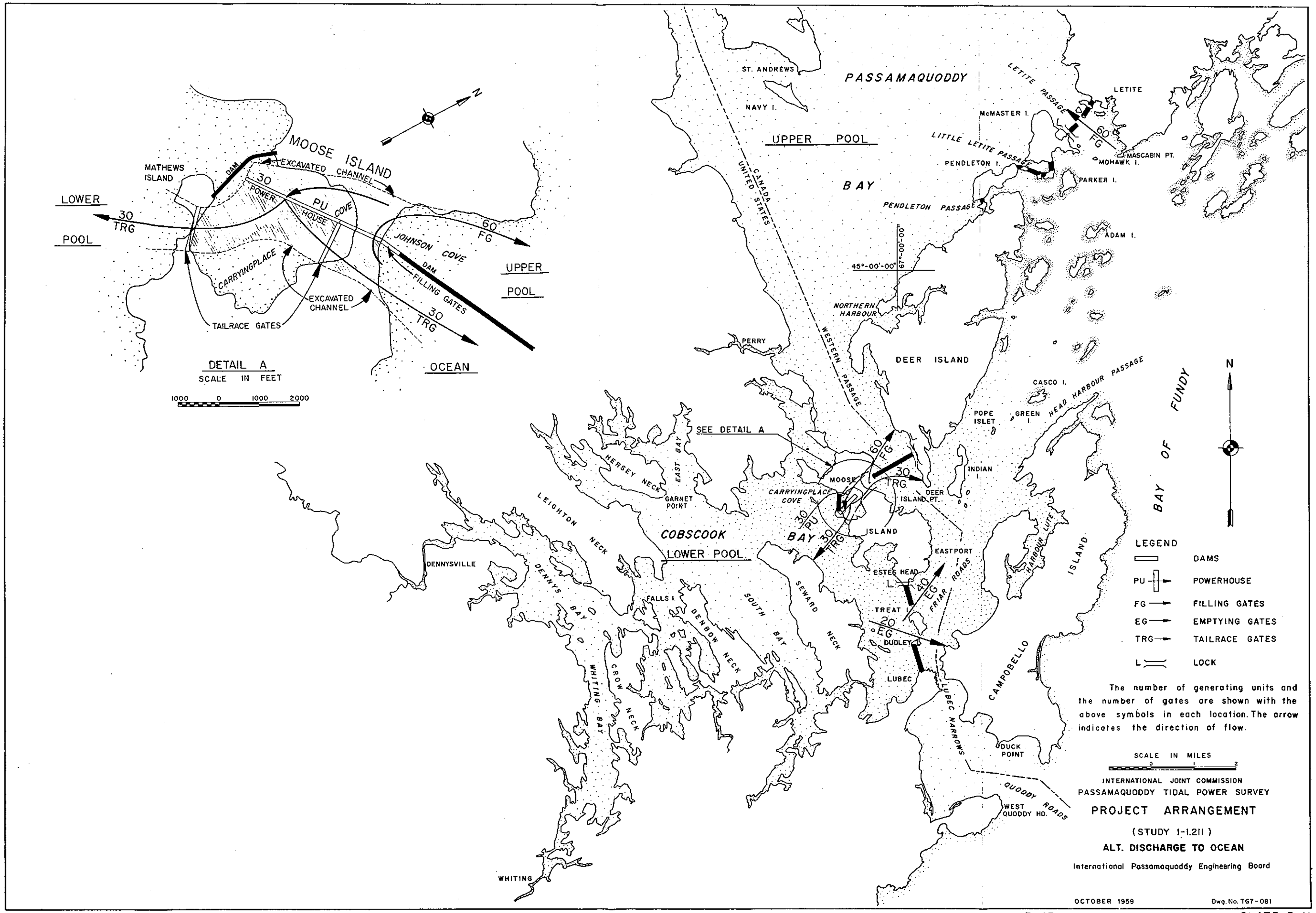
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY

PROJECT ARRANGEMENT
(STUDY 7B-7.22)
TREAT ISLAND PLAN I

International Passamaquoddy Engineering Board

OCTOBER 1959 Dwg. No. TG7-080

PLATE 5-10


MOOSE ISLAND
MATHEWS ISLAND
DAM
EXCAVATED CHANNEL
30 POWER HOUSE
PU
COVE
LOWER POOL
30 TRG
CARRYINGPLACE
60 FG
JOHNSON COVE
DAM
FILLING GATES
30 TRG
UPPER POOL
EXCAVATED CHANNEL
TAILRACE GATES
OCEAN
DETAIL A
SCALE IN FEET
1000 0 1000 2000
ST. ANDREWS
PASSAMAQUODDY
NAVY I.
UPPER POOL
BAY
CANADA
UNITED STATES
LETITE PASSAGE
LETITE
McMASTER I.
LITTLE LETITE PASSAGE
PENDLETON I.
MASCABIN PT.
MOHAWK I.
PARKER I.
PENDLETON PASSAGE
ADAM I.
45°-00'-00"
67°-00'-00"
NORTHERN HARBOUR
WESTERN PASSAGE
PERRY
DEER ISLAND
CASCO I.
HEAD HARBOUR PASSAGE
POPE ISLET
GREEN I.
BAY OF FUNDY
N
SEE DETAIL A
EAST BAY
HERSEY NECK
GARNET POINT
LEIGHTON NECK
CARRYINGPLACE COVE
MOOSE ISLAND
60 FG
30 TRG
DEER ISLAND PT.
INDIAN
30 PU
30 TRG
COBSCOOK BAY
LOWER POOL
DENNYSVILLE
DENNYS BAY
FALLS I.
DENBOW NECK
SOUTH BAY
SEWARD NECK
EASTPORT
ESTES HEAD
L
40 EG
FRIAR ROADS
TREAT I.
20 EG
DUDLEY I.
LUBEC
HARBOUR LITE
ISLAND
CAMPOBELLO
LUBEC NARROWS
WHITING BAY
CROW NECK
DUCK POINT
QUODDY ROADS
WEST QUODDY HD.
WHITING
LEGEND
DAMS
PU POWERHOUSE
FG FILLING GATES
EG EMPTYING GATES
TRG TAILRACE GATES
L LOCK
The number of generating units and the number of gates are shown with the above symbols in each location. The arrow indicates the direction of flow.
SCALE IN MILES
0 1 2
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
PROJECT ARRANGEMENT
(STUDY 1-1.211)
ALT. DISCHARGE TO OCEAN
International Passamaquoddy Engineering Board
OCTOBER 1959
Dwg. No. TG7-081

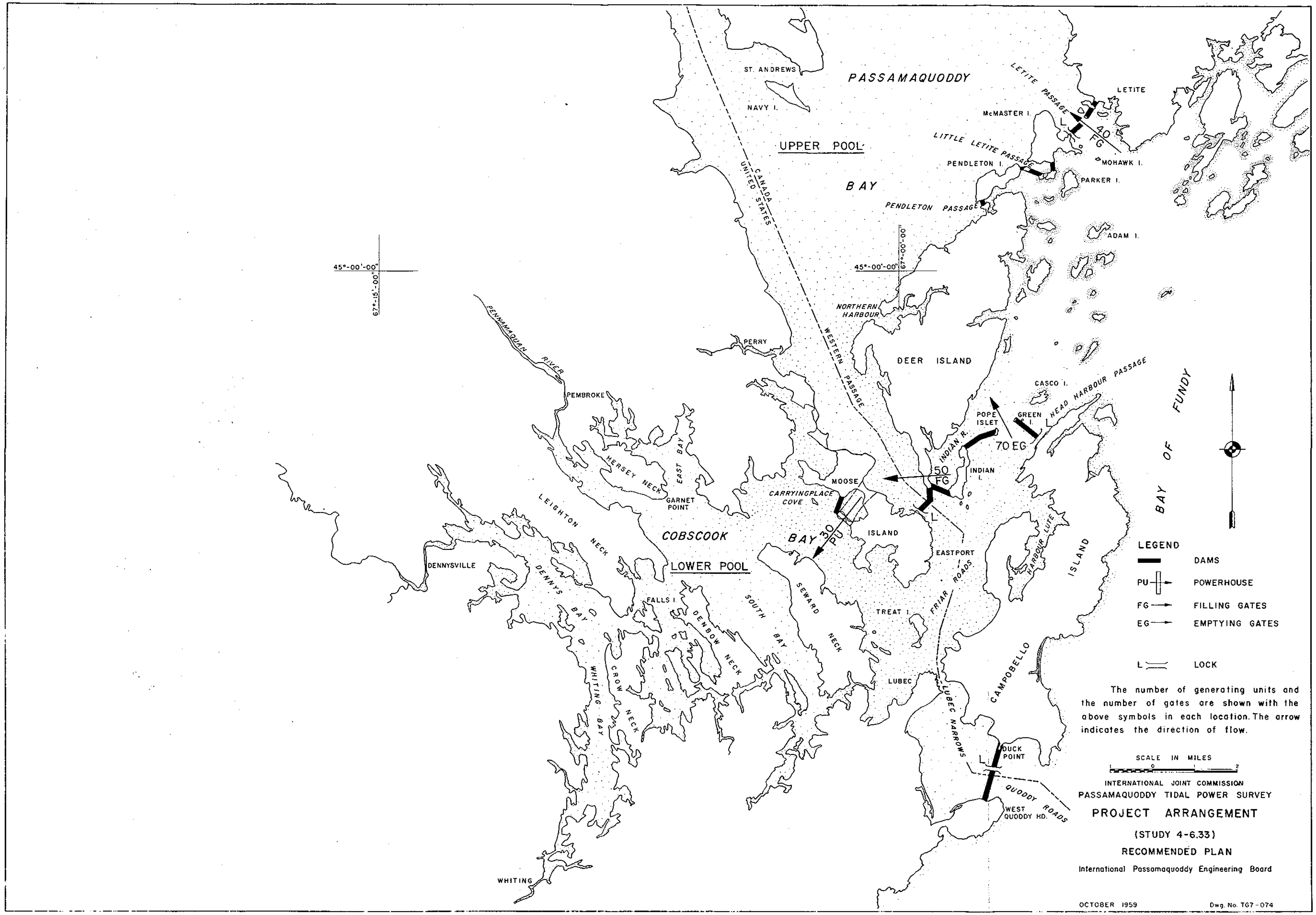

ST. ANDREWS
PASSAMAQUODDY
NAVY I.
UPPER POOL
BAY
CANADA
UNITED STATES
LETITE PASSAGE
LETITE
McMASTER I.
L
40
FG
LITTLE LETITE PASSAGE
MOHAWK I.
PENDLETON I.
PARKER I.
PENDLETON PASSAGE
ADAM I.
45°-00'-00"
67°-00'-00"
67°-15'-00"
NORTHERN HARBOUR
WESTERN PASSAGE
DEER ISLAND
PERRY
PENNAMAQUAN RIVER
PEMBROKE
CASCO I.
HEAD HARBOUR PASSAGE
BAY OF FUNDY
POPE ISLET
GREEN I.
INDIAN R.
70 EG
50
FG
INDIAN I.
HERSEY NECK
EAST BAY
MOOSE
CARRYINGPLACE COVE
GARNET POINT
LEIGHTON NECK
COBSCOOK
BAY
30
PU
ISLAND
EASTPORT
LOWER POOL
DENNYSVILLE
DENNYS BAY
FALLS I.
DENBOW NECK
SOUTH BAY
SEWARD NECK
TREAT I.
FRIAR ROADS
HARBOUR LUTE
ISLAND
CAMPOBELLO
WHITING BAY
CROW NECK
LUBEC
LUBEC NARROWS
DUCK POINT
WEST QUODDY HD.
QUODDY ROADS
WHITING
LEGEND
DAMS
PU POWERHOUSE
FG FILLING GATES
EG EMPTYING GATES
L LOCK
The number of generating units and the number of gates are shown with the above symbols in each location. The arrow indicates the direction of flow.
SCALE IN MILES
1 0 2
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
PROJECT ARRANGEMENT
(STUDY 4-6.33)
RECOMMENDED PLAN
International Passamaquoddy Engineering Board
OCTOBER 1959
Dwg. No. TG7-074

PLATE 5-12

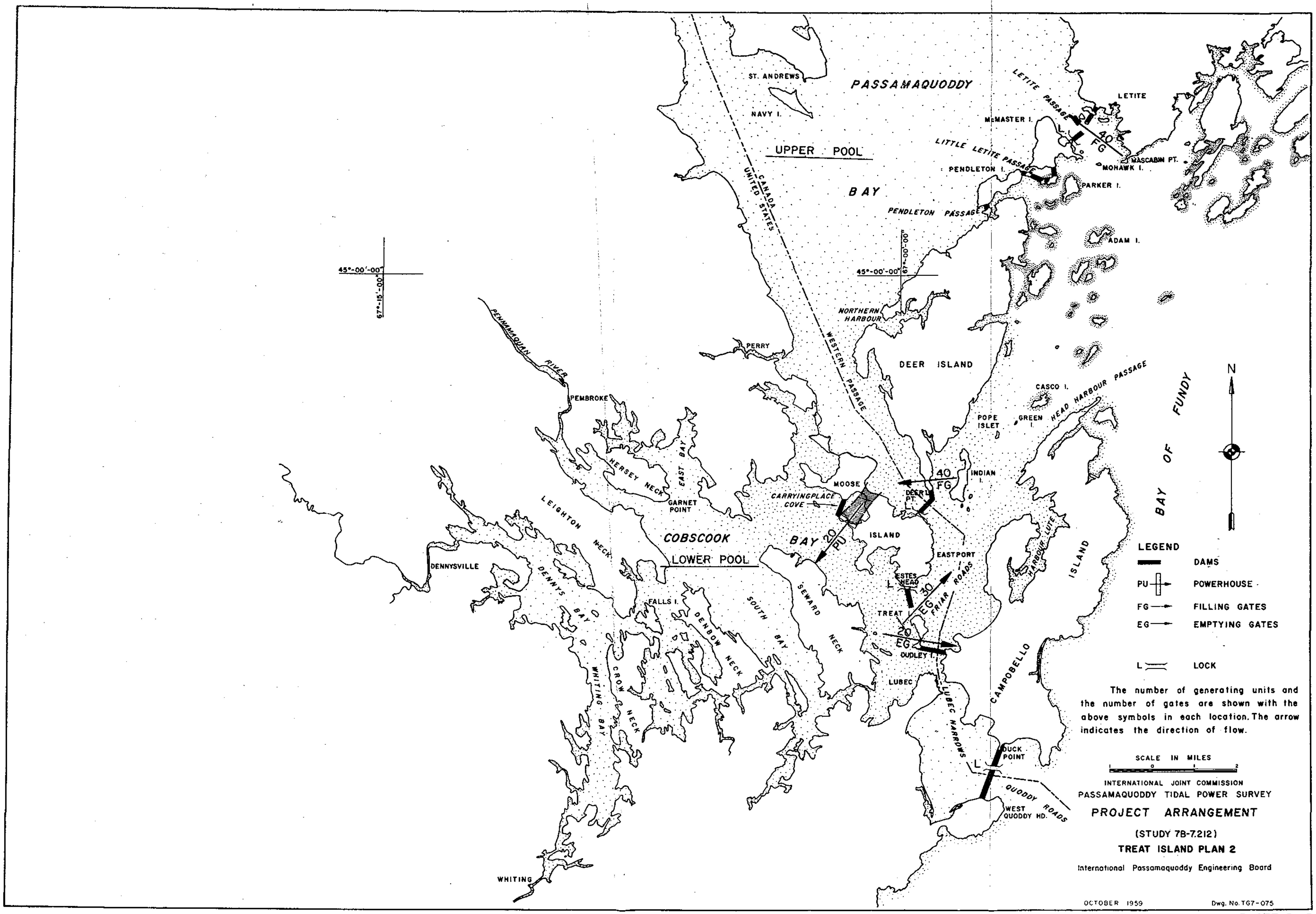

PASSAMAQUODDY
UPPER POOL
BAY
ST. ANDREWS
NAVY I.
CANADA
UNITED STATES
LETITE PASSAGE
LETITE
McMASTER I.
40
FG
LITTLE LETITE PASSAGE
PENDLETON I.
MASCABIN PT.
MOHAWK I.
PARKER I.
PENDLETON PASSAGE
ADAM I.
45°-00'-00"
67°-00'-00"
67°-15'-00"
NORTHERN HARBOUR
WESTERN PASSAGE
PERRY
DEER ISLAND
PENNAMAQUAN RIVER
PEMBROKE
CASCO I.
POPE ISLET
GREEN I.
HEAD HARBOUR PASSAGE
BAY OF FUNDY
N
HERSEY NECK
EAST BAY
INDIAN I.
DEER I. PT.
MOOSE
CARRYINGPLACE COVE
GARNET POINT
LEIGHTON NECK
COBSCOOK
BAY
20
PU
ISLAND
LOWER POOL
EASTPORT
DENNYSVILLE
ESTES HEAD
L
30
EG
FRIAR ROADS
HARBOUR LUTE
ISLAND
DENNYS BAY
FALLS I.
SEWARD NECK
SOUTH BAY
DENBOW NECK
TREAT
DUDLEY I.
LUBEC
CAMPOBELLO
WHITING BAY
CROW NECK
LUBEC NARROWS
DUCK POINT
QUODDY ROADS
WEST QUODDY HD.
WHITING
LEGEND
DAMS
POWERHOUSE
FILLING GATES
EMPTYING GATES
LOCK
The number of generating units and the number of gates are shown with the above symbols in each location. The arrow indicates the direction of flow.
SCALE IN MILES
1 0 1 2
INTERNATIONAL JOINT COMMISSION
PASSAMAQUODDY TIDAL POWER SURVEY
PROJECT ARRANGEMENT
(STUDY 7B-7.212)
TREAT ISLAND PLAN 2
International Passamaquoddy Engineering Board
OCTOBER 1959
Dwg. No. TG7-075

PLATE 5-13